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	Engineering and Design DESIGN OF BREAKWATERS AND JETTIES	
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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-2-2904

DAEN-CWH-D

Engineer Manual
No. 1110-2-2904

8 August 1986

Engineering and Design
DESIGN OF BREAKWATERS AND JETTIES

1. Purpose. This manual provides current guidance and engineering procedures for the design of breakwaters and jetties.

2. Applicability. This manual applies to all HQUSACE/OCE elements and field operating activities (FOA) having responsibility for the design of civil works projects.

3. General. Design considerations for breakwaters and jetties are discussed in this manual. The goal of a good design is to provide an effective structure at a minimal cost, with consideration given to the social and environmental effects.

FOR THE COMMANDER:

ARTHUR E. WILLIAMS
Colonel, Corps of Engineers
Chief of Staff

This manual supersedes EM 1110-2-2904 dated 30 April 1963.

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Table of Contents

CHAPTER 1

INTRODUCTION

1-1. Purpose. This manual provides guidance and engineering procedures for the design of breakwaters and jetties.

1-2. Applicability. This manual applies to all HQUSACE/OCE elements and field operating activities (FOA) having responsibility for the design of civil works projects.

1-3. References. In addition to the design guidance presented herein, additional information on specific subjects can be obtained from the following manuals and special report:

a. ER 1110-2-100

b. ER 1110-2-8151

c. ER 1165-2-304

d. EM 1110-1-1804

e. EM 1110-1-2101

f. EM 1110-2-1607

g. EM 1110-2-1612

h. EM 1110-2-1614

i. EM 1110-2-1615

j. EM 1110-1-1802

k. EM 1110-2-1901

l. EM 1110-2-1902

m. EM 1110-2-1903

n. EM 1110-2-1904

o. EM 1110-2-2000

p. EM 1110-2-2502

q. EM 1110-2-2906

r. EM 1110-2-3300

s. EM 1110-2-5025

t. CEGS 02362

u. CEGS 02366

v. Coastal Engineering Research Center, CE, 1983, "Construction Materials For Coastal Structures," Special Report No. 10, U.S. Army Engineer Waterways Experiment Station, P.O. Box 631, Vicksburg, MS 39180

w. Coastal Engineering Research Center, CE, 1984, "Shore Protection Manual," Vols I and II, U.S. Army Engineer Waterways Experiment Station, P.O. Box 631, Vicksburg, MS 39180. Available from Superintendent of Documents, U.S. Government Printing Office, Washington, DC 20402

1-4. Bibliography. Item numbers are used throughout this manual to indicate bibliographic references. In publications where authors are not listed the organization and the date of publication are given. These publications are listed in alphabetical order in Appendix A and are available for loan upon request to the Technical Information Center (TIC) Library, US Army Engineer Waterways Experiment Station (WES), PO Box 631, Vicksburg, Mississippi 39180-0631.

1-5. Background. The Corps of Engineers is responsible for over 600 breakwaters and jetties, many of which date to the mid and late 1800's. A summary of their locations and types is presented in Appendix D. Originally, the design and the construction of breakwaters and coastal protection structures were based on trial and error resulting from man's conflicts with nature. Later, existing experience was the guiding hand and it was not until the 1930's that model tests were introduced to aid in the design of such structures. Today, model tests are commonly used and play a significant role in the design of sophisticated coastal structures.

1-6. Inventory. An inventory of WES breakwater stability studies is given in Appendix B.

1-7. Symbols. For convenience, symbols and unusual abbreviations used in this manual are listed and defined in the Notation (Appendix C).

1-8. General. This manual presents factors that influence the location of breakwaters and jetties, the determination of the type and magnitude of forces to which the structures will be subjected, the selection of construction materials, and the choice of structure types that best suit a particular location. Even though design methodologies are based on the latest state-of-the-art developments, they are not intended to replace individual engineering initiative. Departures from the manual which are in accordance with sound engineering principles and judgment are acceptable for unusual situations;

however, to prevent misunderstanding between the designer and reviewer those departures should be explained and supported. This manual presents guidance for the design of breakwaters and jetties; however, the guidance herein is applicable to other coastal structures that are subjected to similar forces. Typical examples of various types of existing breakwaters and jetties and the experience gained from their performance are included within this manual.

1-9. Definitions. The following definitions and distinctions are offered for the sake of clarity:

a. Breakwater. A breakwater is a structure employed to reflect and/or dissipate the energy of water waves and thus prevent or reduce wave action in an area it is desired to protect. Breakwaters for navigation purposes are constructed to create sufficiently calm waters in a harbor area, thereby providing protection for the safe mooring, operating, and handling of ships and protection of shipping facilities. Breakwaters are sometimes constructed within large, established harbors to protect shipping and small craft in an area that would be exposed to wave action excessive for the purpose. Offshore breakwaters may serve as aids to navigation or shore protection or as both, and differ from other breakwaters in that they are generally parallel to and not connected with the shore.

b. Jetty. A jetty is a structure, generally built perpendicular to the shore, extending into a body of water to direct and confine a stream or tidal flow to a selected channel and to prevent or reduce shoaling of that channel. Jetties at the entrance to a bay or a river also serve to protect the entrance channel from storm waves and crosscurrents, and when located at inlets through barrier beaches jetties also serve to stabilize the inlet location.

c. Stone and Rock. Stone is defined as a construction material; that is, rock which has been removed from its natural position. Rock is defined as a naturally formed consolidated mineral matter in its natural geological position.

1-10. Types of Breakwaters and Jetties.

a. Rubble-Mound. Rubble-mound structures are typically constructed with a core of quarry-run stone, sand, or slag, and protected from wave action by one or more stone underlayers and a cover layer composed of stone or specially shaped concrete armor units. The structures are suitable for nearly all types of foundations and any economically acceptable water depth. A proposed structure may necessarily be designed for either nonbreaking or breaking waves, depending upon positioning of the breakwater and severity of anticipated wave action during its economic life. Some local wave conditions may be of such magnitude that the protective cover layer must consist of specially shaped concrete armor units in order to provide economic construction of a stable breakwater. Most local design requirements are advantageously met by stone armor. Figure 1-1 shows a typical rubble-mound section. The design of rubble-mound structures is discussed in Chapter 4.

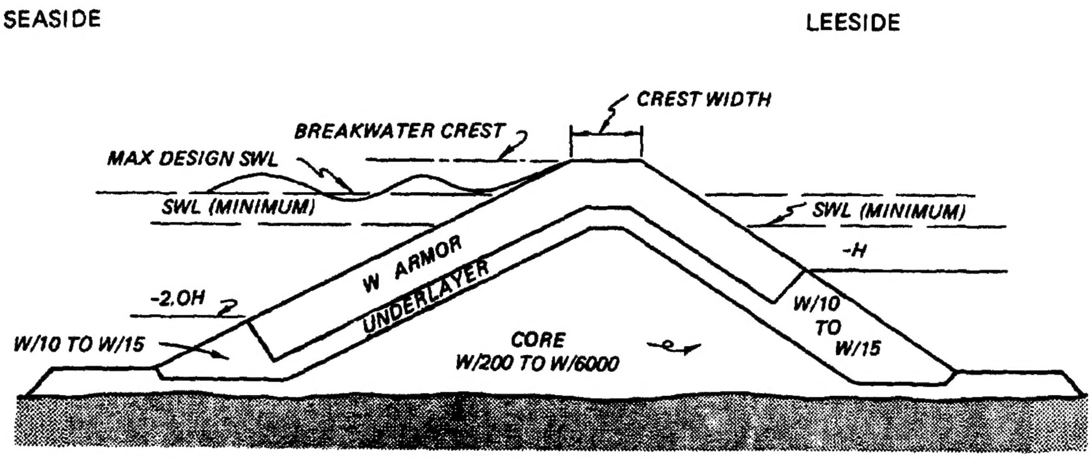


Figure 1-1. Typical rubble-mound section for seaside wave exposure

b. Sheet Piling. Timber sheet piling, held in position by round timber piles and usually protected at the base by stone riprap, has been used where storm waves are mild. Timber used in salt water where marine borers are present should be treated to avoid premature deterioration of the structure; timber pile structures are also subject to sand and ice abrasion. Steel and concrete sheet piling are also used; compared with timber structures, steel and concrete generally have higher initial costs and lower maintenance costs. Figures 1-2 through 1-4 present examples of timber, concrete, and steel structures, respectively. The design of sheet pile structures is discussed in Chapter 5.

c. Floating. Any structure which has a composite unit weight less than the water in which the structure is placed and is primarily used to reduce wave heights can be categorized into this group. Typically, floating structures are only effective for relatively short wave periods. Some advantages include portability, low cost, insensitivity to water depth, and possible enhancement of marine life. These structures can be box, pontoon, tethered float, or a variety of other types. Examples of the most commonly used types are shown in figure 1-5. Design of pontoon and tethered-float scrap tire breakwaters is described in Chapter 6.

d. Miscellaneous. Other types of structures that do not fit into the previous categories are as follows:

(1) Crib. Structures of this type are built of timber or precast concrete members, and some of the compartments of the crib are floored. The timber cribs are floated into position and settled upon a prepared foundation by filling the floored compartments with stone. The unfloored compartments are then filled with stone to give stability. The structure is capped with

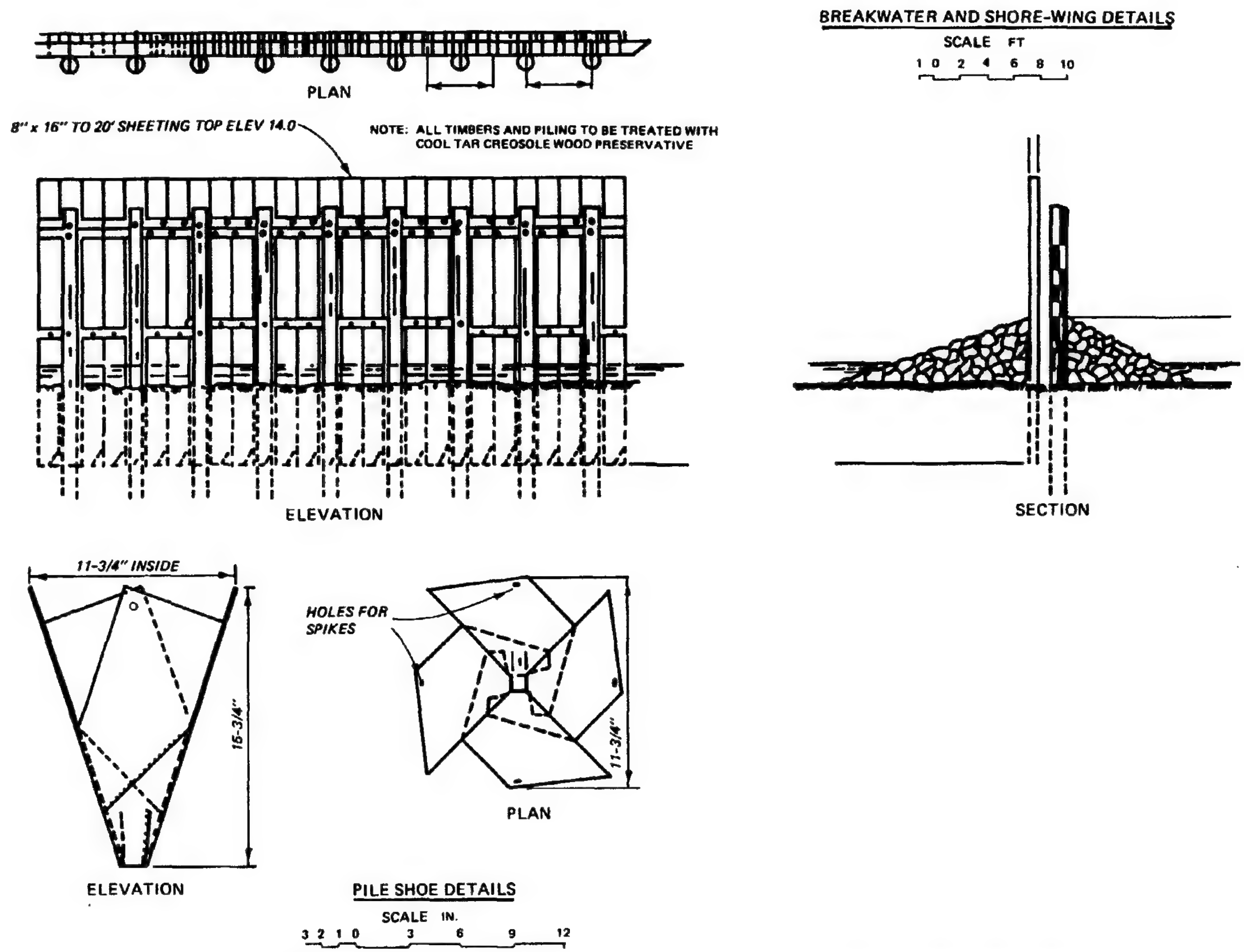
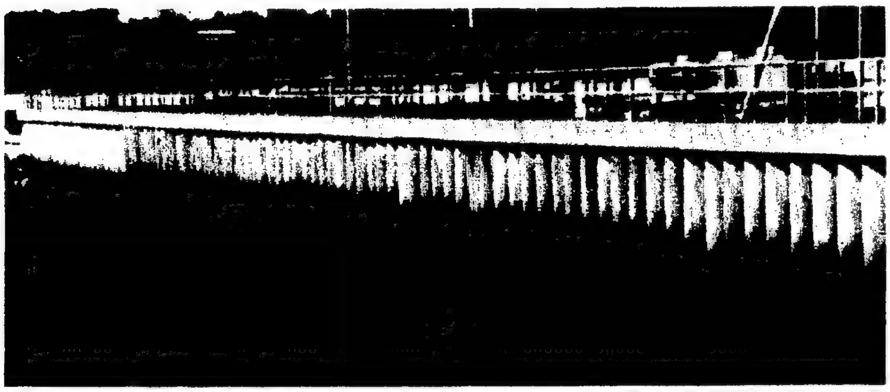
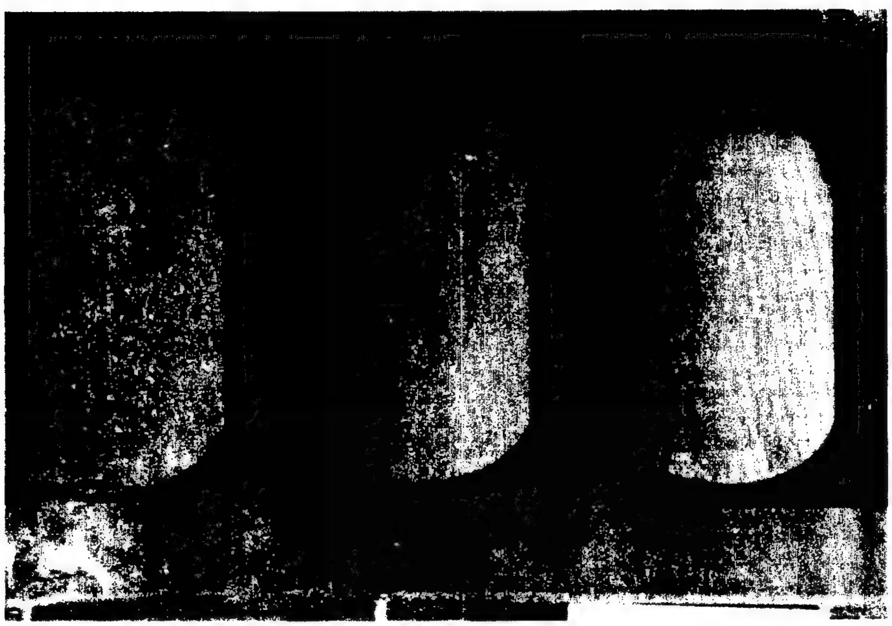


Figure 1-2. Timber sheet pile breakwater constructed at Yaquina Bay and Harbor, Oregon



a. General view



b. Close-up

Figure 1-3. Concrete breakwater constructed at Pass Christian, Mississippi

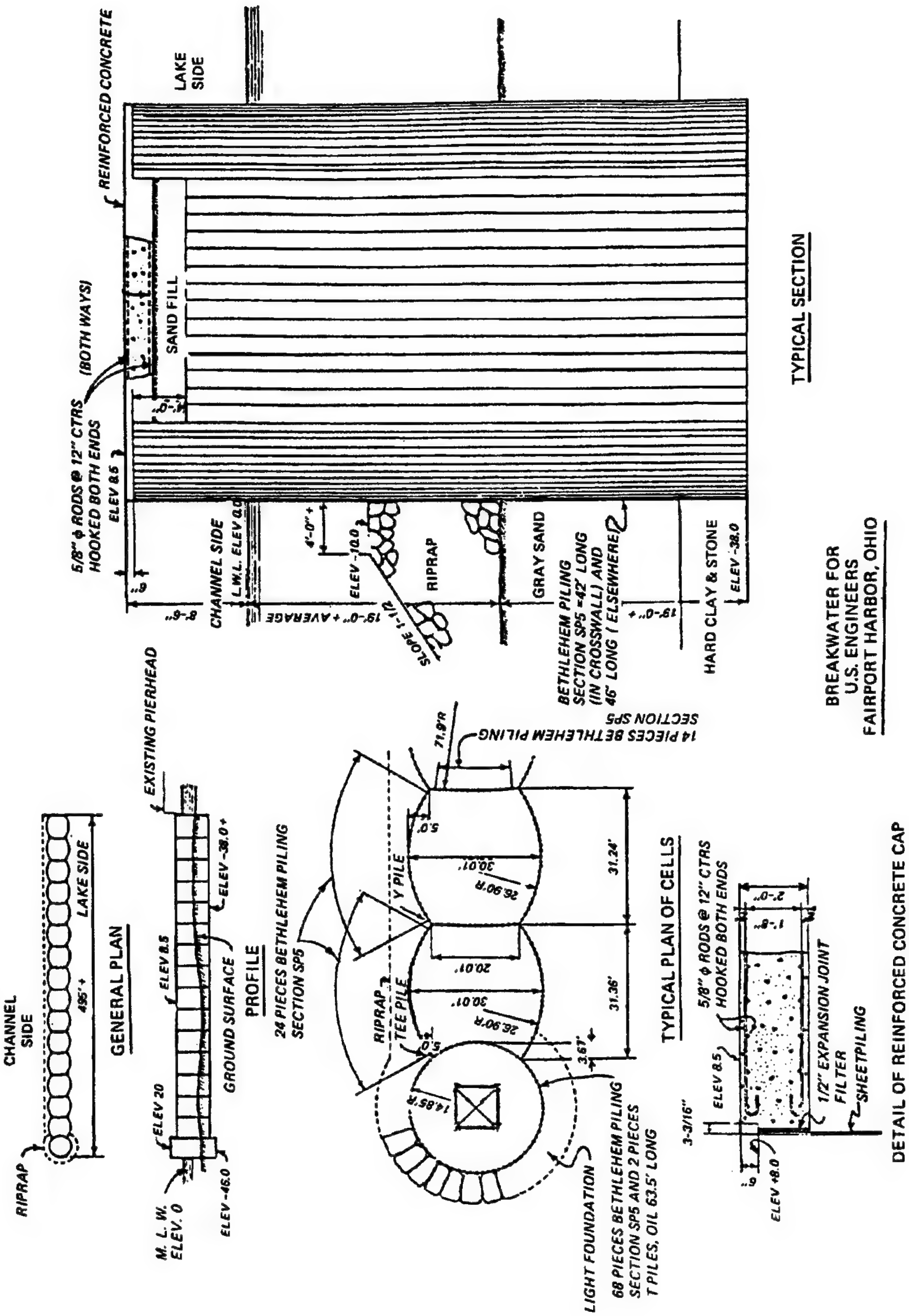


Figure 1-4. Steel sheet pile breakwater constructed at Fairport Harbor, Ohio

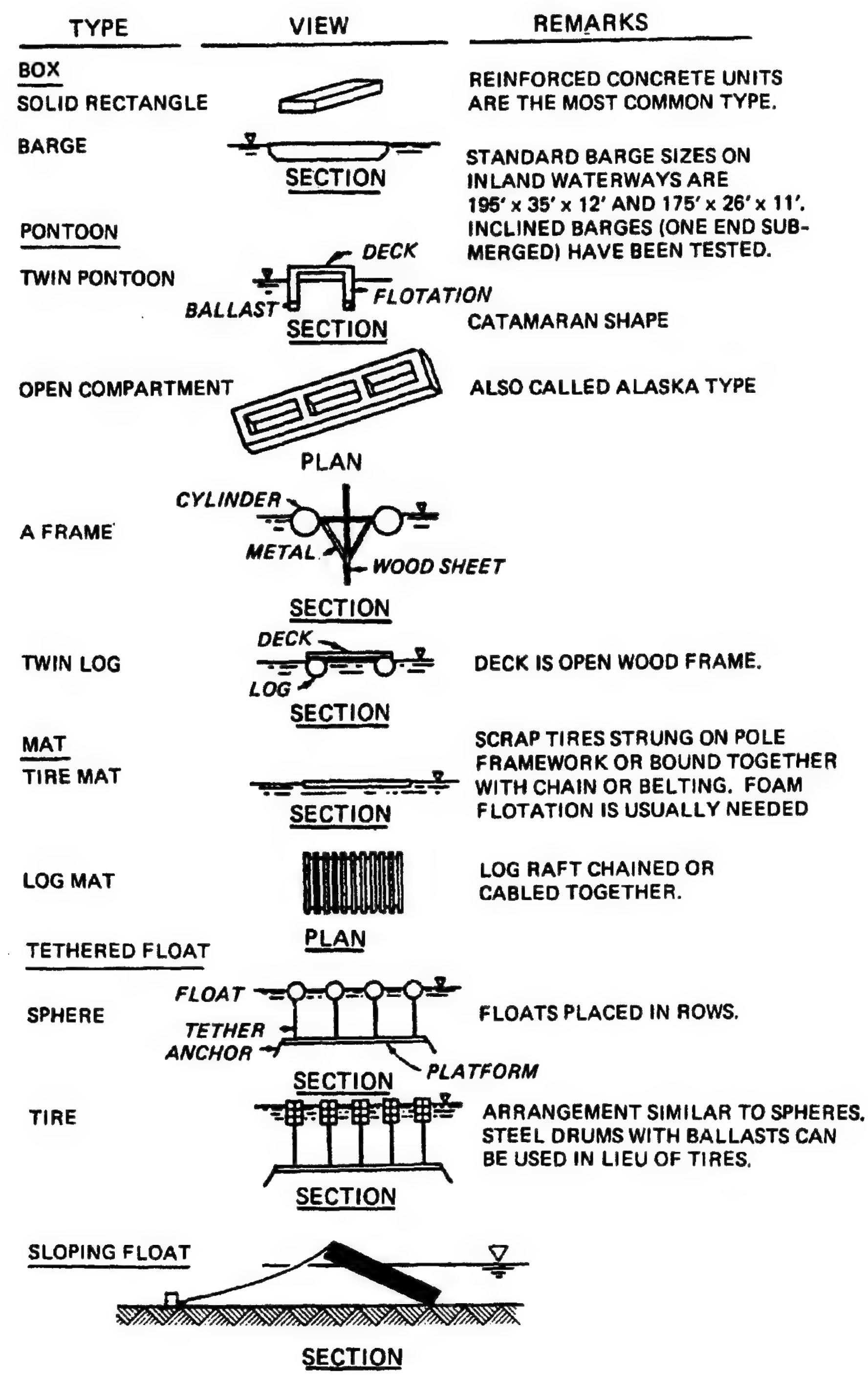


Figure 1-5. Types of floating breakwaters

timber, concrete, or capstones. Stone-filled timber cribs can withstand considerable settlement and racking without rupture. The superstructure and decking of cribs set on a rubble-mound foundation are often constructed of timber to allow for settlement of the crib. Timber used in this construction in salt water must be treated for protection against the marine borer. When decay of the timber makes replacement of the superstructure necessary, concrete can beused since the structure will probably have settled into a permanent position by that time. An example of a timber crib breakwater is shown in figure 1-6.

(2) Composite. Monolithic walls placed on underwater rubble mounds are referred to as composite breakwaters in this manual. The rubble mound is generally used either as a foundation for the wall or as a main substructure surmounted by a wall superstructure with a vertical or inclined face. It is often used where the foundation is soft and subject to scour. The foundation is usually prepared by placing layers of rubble until adequate bearing pressure is obtained for the complete structure. Figure 1-7 shows examples of typical composite jetty sections.

(3) Concrete caisson. Caisson construction is sometimes used whereby reinforced concrete shells are floated into position, settled upon a prepared foundation, filled with stone or sand to give stability, and then capped with concrete slabs or capstones. Such breakwaters can be constructed with parapet walls. Concrete caissons are generally of two types: one type has a bottom of reinforced concrete which is an integral part of the caisson; the other type is not provided with a permanent bottom. The bottom opening of this latter type is closed with a temporary wooden bottom which is removed after the caisson is placed on the foundation. The stone used to fill the compartments combines with the foundation material to provide additional resistance against horizontal movement. Typical sections of concrete caisson breakwaters are shown in figure 1-8.

(4) Pneumatic. The pneumatic breakwater is composed of a bubble screen generated by releasing compressed air from a submerged manifold. Rising bubbles induce a vertical current, which in turn produces horizontal currents away from the bubble-screen area on or near the water surface in both directions; i.e., in the direction of oncoming waves and in the opposite direction. Near bottom, the corresponding currents flow toward the bubble screen, thus completing the circulation pattern. Surface currents moving against the direction of wave propagation produce some attenuation of the waves; however, this type of breakwater can only effect a partial dissipation of the incident wave energy. It becomes more effective as the wave steepness (H/L) and the relative depth (d/L) increase (short-period waves in deep water). Figure 1-9 shows a conceptual sketch. Pneumatic breakwaters are discussed further in Chapter 7.

(5) Hydraulic. Hydraulic breakwaters dissipate incident wave energy by directing a current against the oncoming waves. Currents are generated by water jets from a manifold system located at or near the water surface. This

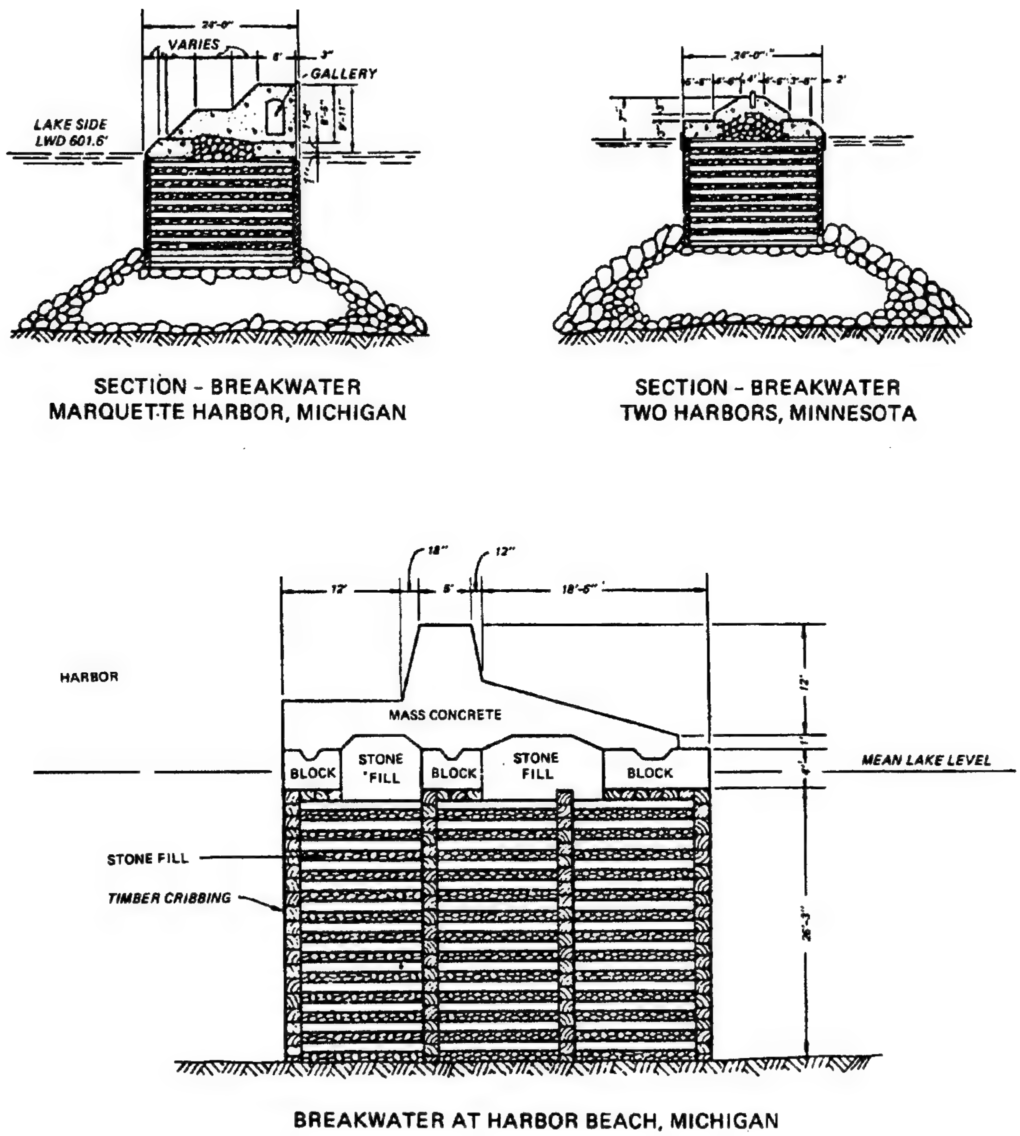


Figure 1-6. Examples of timber crib breakwaters constructed on the Great Lakes

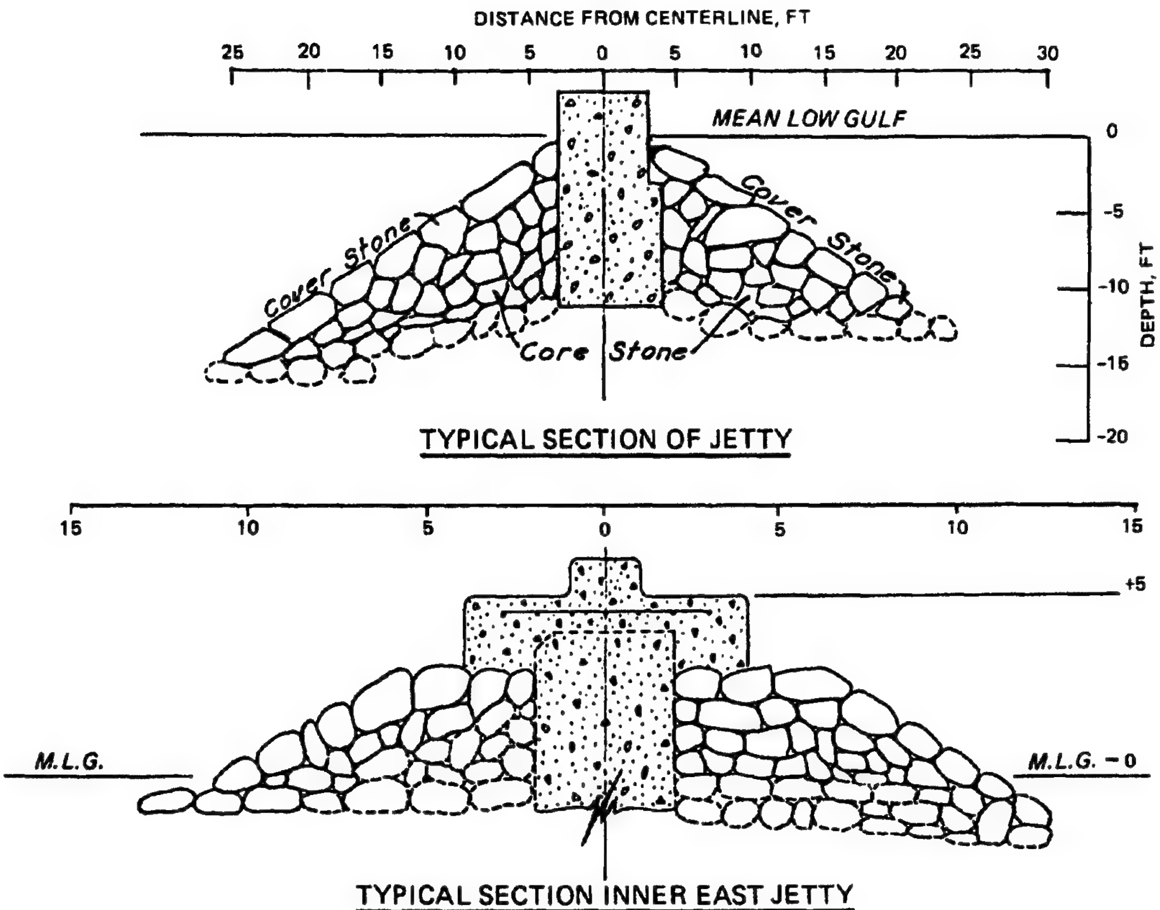
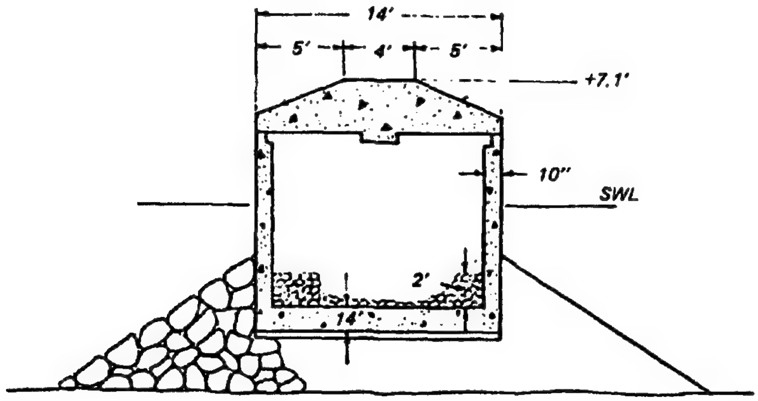
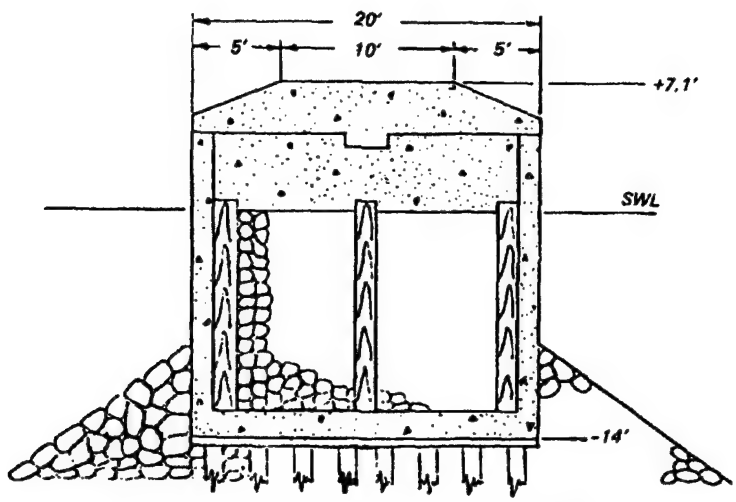


Figure 1-7. Composite jetty sections constructed at South and Southwest Passes, Mississippi River Outlets



TYPICAL SECTION - NORTH AND SOUTH BREAKWATER
MILWAUKIE HARBOR WISCONSIN



TYPICAL SECTION - NORTH BREAKWATER
SHEBOYGAN HARBOR, WISCONSIN

Figure 1-8. Examples of concrete caisson breakwaters constructed on the Great Lakes

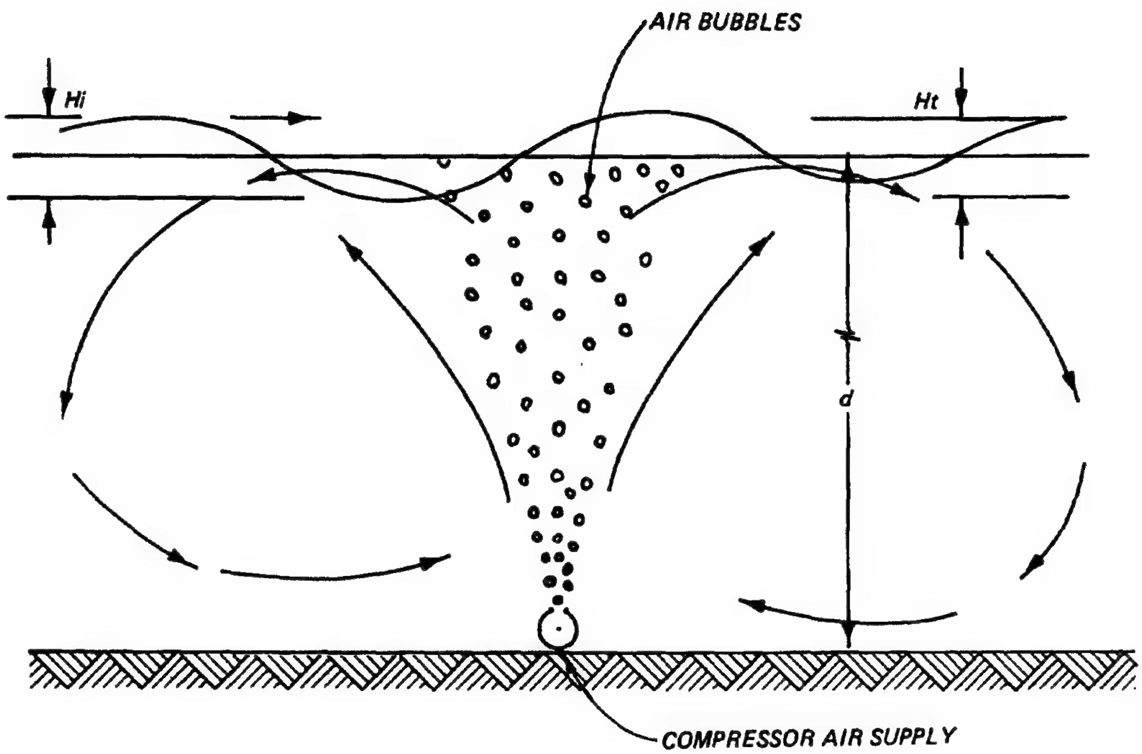


Figure 1-9. Conceptual sketch of the pneumatic breakwater

method of achieving wave-height reduction by the use of countercurrents is the same for pneumatic and hydraulic breakwaters. Thus, the practical limitations are the same; i.e., the range of wave conditions for which adequate wave reduction can be achieved is limited to short-period waves in relatively deep water. A conceptual sketch is shown in figure 1-10. Hydraulic breakwaters are discussed further in Chapter 7. It should be noted that neither pneumatic nor hydraulic breakwaters have been field proven.

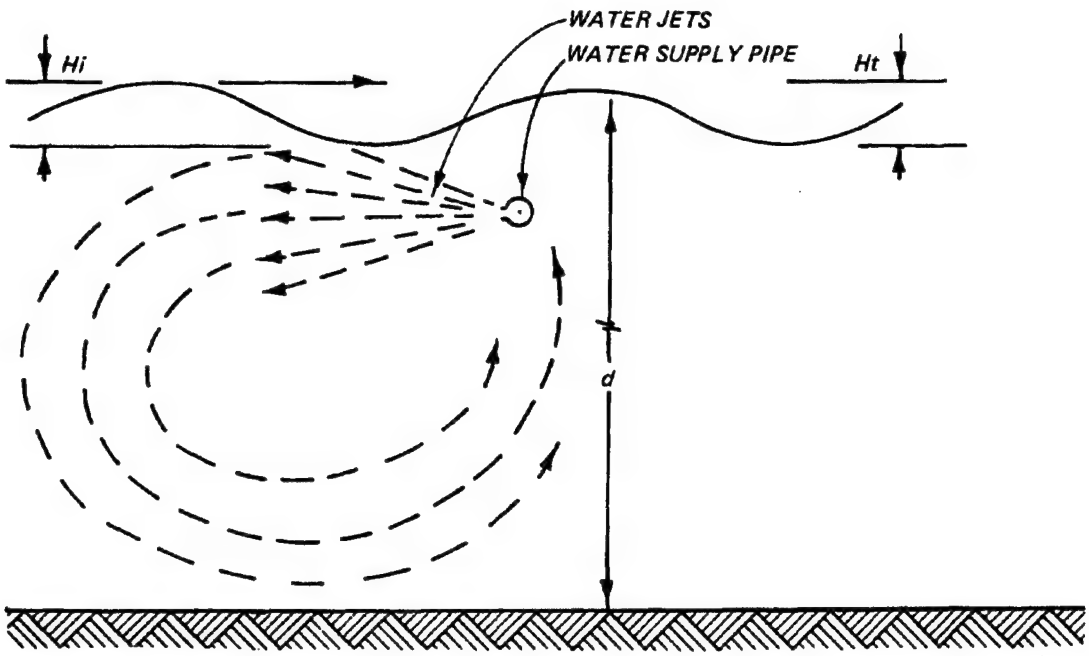


Figure 1-10. Conceptual sketch of the hydraulic breakwater

(6) Sloping float. The sloping float breakwater (SFB) is a wave barrier that consists of a row of flat slabs or panels. The weight distribution of these slabs or panels is such that each panel rests with one end above the water surface and the other end on or near the bottom. Various types of construction are possible; however, compartmentalized steel or concrete barges are the most practical. The height of protrusion of the bow above the water surface (i.e., the freeboard) is controlled by flooding a selected number of pontoons. Barge modules are sited so the bow faces the primary direction of wave attack, and wave attenuation is achieved by reflection and turbulent dissipation. Figure 1-11 shows a conceptual sketch. SFB's are discussed further in Chapter 7.

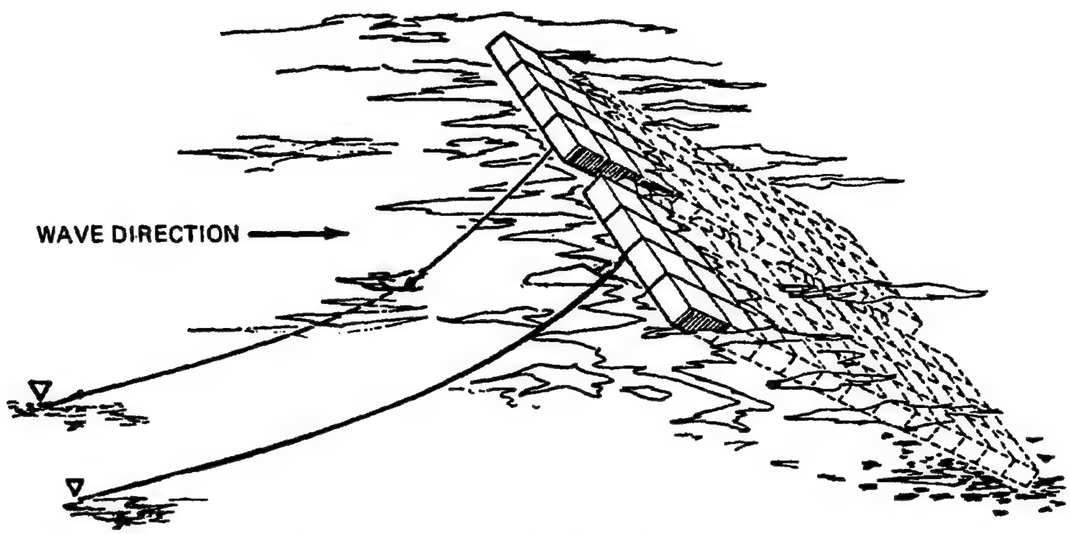


Figure 1-11. Conceptual sketch of the SFB

CHAPTER 2

FUNCTIONAL DESIGN

2-1. Design Overview.

a. General.

(1) The selection and the evaluation of site conditions and hydraulic factors are necessary for the functional planning of the structure and the selection of design conditions. Because of local site conditions, it may be impractical to evaluate alternative structure types. For example, foundation conditions may eliminate a gravity structure, the size and location of the area to be protected may dictate the orientation and shape of the structure, or the longshore transport rate may necessitate supplementary structures to minimize channel maintenance and control adverse effects on adjacent shores. The design reports should provide sufficient information to justify the recommended design and adequate presentation of alternatives to assure that all practical structural and nonstructural options were considered. Design memoranda should include the formulas used, the assumptions made, and the evaluation of coefficients, so the reviewer can check any particular computation needed to verify the design. Refraction and diffraction diagrams should be included in the design memoranda. Deviation from or modification of accepted practices should be explained and substantiated. The design memoranda will include also an evaluation of the environmental aspects of the recommended plan and each of the alternatives.

(2) The cost of construction is generally a controlling factor in determining the type of structure to be used. A limited number of types of construction will be practical in any locality; but the cost of constructing and maintaining the different types may vary considerably, and the final decisions in design will be dictated by either the initial cost of the structure or the annual costs. A comparison should be made on the basis of annual cost which includes the interest, amortization, and maintenance. Comparative designs of several types with estimates of annual costs are necessary before final decisions can be made. Annual costs of maintaining the navigation channel and other associated costs, such as any costs incurred by the mitigation of anticipated unwanted effects on adjacent shores, are items for consideration.

(3) The quantities of material required for breakwater or jetty construction usually are large, and considerable savings in transportation cost may be achieved if suitable materials can be obtained locally. The selection of a rubble-mound-type structure is generally dependent upon the availability of a large amount of suitable stone at low cost, and the use of concrete will be affected by the availability of quality aggregates.

(4) The average annual cost of maintenance is often a significant portion of the total annual cost of a project. However, a structure designed to resist the action or stresses of moderate storms, but which may suffer some

damage without complete destruction in a severe storm, may show a lower total annual cost than a structure designed to be completely stable for all storms. The lowest annual cost should be determined by considering the annual cost for increments of stability.

b. Design Verification. The formulas and design charts presented in this manual can be used in the preliminary design to screen alternatives. Existing long-term prototype data and/or prototype tests can also be a part of design verification. However, final designs may require verification by hydraulic model testing. Model tests can evaluate armor stability, wave runup and transmission, and potential effects on adjacent shorelines.

c. Monitoring. Development of a monitoring plan should be included as a part of the project design. The plan can include periodic surveys and inspections, comparison of survey results with design predictions, and comparison of actual maintenance costs with predicted maintenance costs.

2-2. Design Studies. The design of breakwaters and jetties requires an understanding of the problem, assembly and evaluation of all pertinent facts, and development of a rational plan. The design engineer is responsible for developing the design rationale and sufficient alternative plans so that the economically optimum plan is evident and the recommended plan is substantiated. Applicable Corps of Engineers (Corps) guidance should be considered in the design. Pertinent textbooks, research reports, technical reports, and expertise from other agencies may be used as source information. The steps leading to a sound plan are outlined as follows:

a. Review appropriate Engineer Regulations, Manuals, and Technical Letters and other published information.

b. Assemble and analyze pertinent factors and environmental data.

c. Conduct baseline surveys.

d. Select a rational set of design conditions.

e. Develop several alternative layouts with annual costs

f. Develop an operation and maintenance plan.

g. Select an economically optimum plan.

h. Assess environmental and other impacts.

i. Develop a recommended plan.

2-3. Typical Engineering Studies. The following kinds of studies are normally undertaken for breakwater and jetty design:

a. Water levels and datums.

b. Winds.

c. Waves.

d. Currents.

e. Geotechnical considerations.

f. Construction materials and sources.

g. Ice conditions.

h. Shoreline changes.

i. Prior projects and their effects.

j. Baseline surveys.

k. Constructability.

l. Design life, degree of protection, and design conditions.

m. Dredging and disposal.

n. Seismic design.

o. Vessel impact.

p. Environmental impact.

q. Model tests.

r. Operation and maintenance.

2-4. Water Levels and Datums. Both maximum and minimum water levels are needed for the designing of breakwaters and jetties. Water levels can be affected by storm surges, seiches, river discharges, natural lake fluctuations, reservoir storage limits, and ocean tides. High-water levels are used to estimate maximum depth-limited breaking wave heights and to determine crown elevations. Low-water levels are generally needed for toe design.

a. Tide Predictions. The National Ocean Service (NOS) publishes tide height predictions and tide ranges. Figure 2-1 shows spring tide ranges for the continental United States. Published tide predictions are sufficient for most project designs; however, prototype observations may be required in some instances.

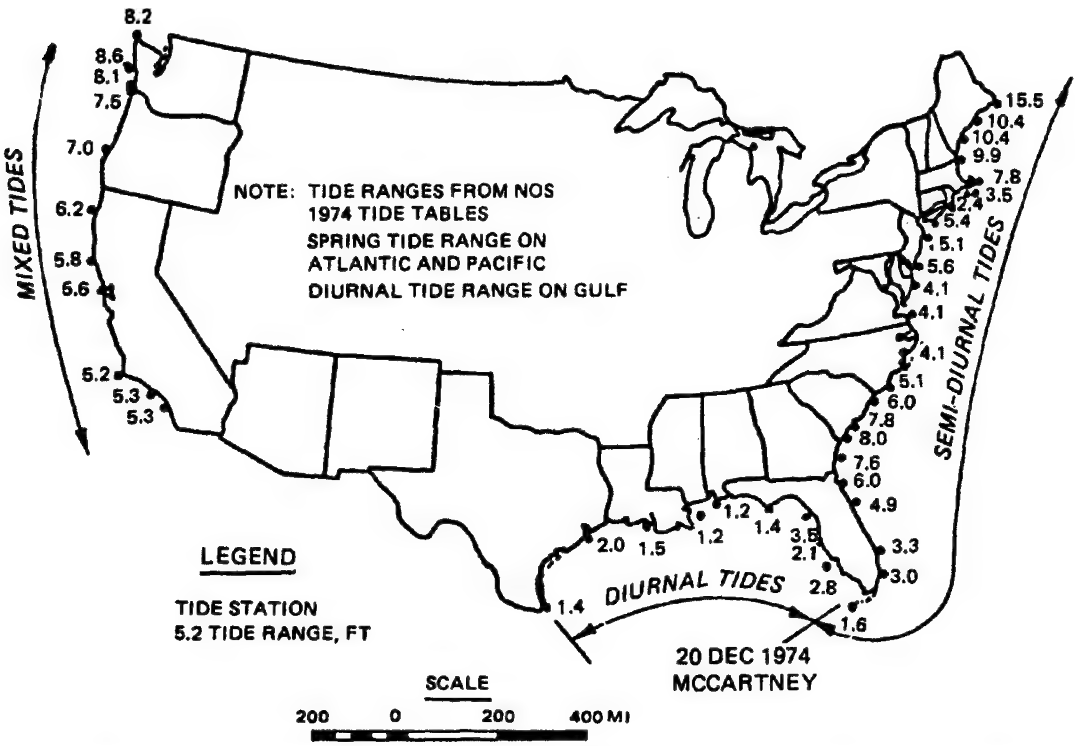


Figure 2-1. Ocean tide ranges for the continental United States

b. Datum Planes. Structural features should be referred to appropriate low-water datum planes. The relationship of low-water datum to the National Geodetic Vertical Datum (NGVD) will be needed for vertical control of construction. The low-water datum for the Atlantic and Gulf Coasts is being converted to mean lower low water (MLLW). Until the conversion is complete, the use of mean low water (MLW) for the Atlantic and Gulf Coast low water datum (GCLWD) is acceptable. Other low-water datums are as follows:

Pacific Coast: Mean lower low water (MLLW)

Great Lakes: International Great Lakes Datum (IGLD)

Rivers: River, low-water datum planes (local)

Reservoirs: Recreation pool levels

2-5. Waves. Naturally occurring wind waves and vessel-generated waves require analysis and prediction. Wave conditions are needed for various elements of the project design.

a. Wind Waves. Prediction of wind wave heights and periods can be made using techniques presented in item 132. Wave information based on numerical hindcasts for some coastal waters and the Great Lakes has been published by the US Army Engineer Waterways Experiment Station (WES) (items 39, 40, 41, 111, 112, 113, 114, and 115). These wave heights and periods are applicable for deep water and require refraction and diffraction analysis to develop wave characteristics at the project site. Chapter 2, item 132, presents a method for calculating refraction and diffraction effects. If feasible, installation of wind and wave gages at the project site is strongly recommended. One year of wind and wave records is considered a minimum to verify or adjust wave predictions before the design is made final.

b. Vessel-Generated Waves. Passing vessels may generate larger waves than the wind. This is particularly true for small boat harbors. The height of waves generated by a moving vessel is dependent on the following:

(1) Vessel speed.

(2) Vessel draft and hull shape.

(3) Water depth.

(4) Blockage ratio of ship-to-channel cross section.

The effects of waves will depend on the height of the wave generated and the distance between the ship and the project site. An estimate of the height of a ship-generated wave can be obtained by assuming that the wave height (crest to trough) will be equal to twice the amount of vessel squat. The wave height at the shore is then computed using refraction and diffraction techniques. The wavelength is equal to approximately one-third of the vessel length. If vessel-generated waves are considered the design wave, model tests or prototype measurements will be needed to verify or adjust the predictions.

c. Tsunami Waves. Tsunami waves can usually be predicted with sufficient accuracy by performing a statistical extrapolation of historical data. However, when the primary purpose of a structure is protection against tsunami waves, it may be necessary to numerically study tsunami generation, propagation, and amplification, and then to apply the results of the study in a physical model to determine tsunami/structure interaction and stability.

d. Selection of Test Waves from Prototype Data. Measured prototype wave data on which a comprehensive statistical analysis of wave conditions can be based are usually unavailable for various project areas. However, statistical or deepwater wave data representative of these areas can sometimes be obtained and transposed to the site by refraction and diffraction calculations. Sources of prototype wave data for the Atlantic, Gulf, and Pacific Coasts are items 11, 41, 84, 85, 94, 100, 136, and 137. Wave data

commonly used for study sites on the Great Lakes can be obtained from items 5, 11, 35, 111, 112, 113, 114, 115, and 127.

2-6. Currents. Currents can be tidal, river, wind, or seiche induced. Prediction of current strength and duration is needed for the selection of design conditions. Current forces and flow velocities are considered in the designing of rubble-mound toes and floating breakwater mooring systems.

2-7. Geotechnical Considerations. The selection of the type of breakwater and jetty structure as well as the configuration is significantly influenced by geotechnical and site conditions. Foundation conditions at a site may range from solid rock to soft mud, and each foundation condition requires different design considerations. Geotechnical studies for a project should include adequate subsurface investigations, laboratory testing, and analyses to insure the adequacy of the design and constructability.

a. Exploration and Testing. Exploration along the proposed alignment shall be made to evaluate the foundations conditions. Exploration includes drilling test holes at appropriate intervals to obtain disturbed and undisturbed samples for classification tests, moisture content, density, and consistency. Representative samples should be obtained for shear and consolidation testing when warranted.

b. Stability. Stability analyses for rubble-mound structures should be performed in accordance with EM 1110-2-1902. Selected strength parameters should be based on laboratory tests representing actual and anticipated field conditions. Both the during construction and long-term stability conditions should be analyzed. As a minimum, longitudinal and transverse sections should be evaluated. In addition, analyses should be performed for special conditions such as temporary construction slopes, anticipated scour, and the location and potential migrations of adjacent channels.

c. Settlement. Total and potential differential settlement, both longitudinal and transverse as well as during and after construction of the breakwater or jetty structure, should be determined in accordance with EM 1110-2-1904. These values should be used in determining the need for crest overbuild as well as the stress and stability of structural elements sensitive to the movement such as the prefabricated armor unit and caisson structures.

d. Foundation Protection. Migration of fines from the foundation may cause settlement and other damage to a structure. This damage can be mitigated by a bedding layer that conforms to the filter requirements. Scour of the foundation can also cause failure of the toe. The zone of scour and the location of stability failure areas should be clearly identified to determine the extent of toe protection.

(1) Rubble aprons. Experience indicates that the use of rubble aprons to protect the foundation of vertical or almost vertical walls from undermining is advisable, except for depths well below twice the maximum wave

height or on seabeds of very hard and durable material, such as ledge rock. If wave action causes a volume of water to spill over the breakwater, the effect of this water is equivalent to the action of water discharging over a dam; and protection of the foundation on the harbor side is as important as on the sea side or lake side.

(2) Bedding layers. When large stone is placed directly on a sand bottom at depths insufficient to avoid wave action and currents on the bottom, it will settle into the sand until it reaches a depth below which the sand will not be disturbed by the currents. Even if the amount of stone deposited is sufficient to provide protection after settlement, the settlement will probably be irregular, resulting in an irregular and unsightly structure which is more susceptible to wave damage. To prevent waves and currents from removing foundation materials through the voids in stone structures or protective aprons and destroying their support, all stone and other materials having large voids should be placed on a bedding layer of smaller stone. This material should be sufficiently graded to prevent the removal of the foundation material through the blanket or the loss of blanket material into the voids of the cover stone.

e. Low Bearing Capacity Foundations. When the bottom material is soft and does not have sufficient bearing capacity to support the structure, a pile foundation may be needed. In preparing a foundation of this type the piles can be driven to a minimum depth by use of a water jet, but below this depth they should be driven by hammer without the use of a water jet until the piles will safely support the design load. After foundation piles have been driven, stone should be deposited over the entire area and, after settlement, leveled to the elevation of the pile tops. If necessary, bottom material between the upper portion of the piles can be removed before the stone is deposited. As an alternative, pile-anchored floating breakwaters may prove feasible, provided that design wave periods are relatively short.

f. Construction Materials. After the stone size has been determined and the type of structure selected, the materials and their sources and availability should be investigated. In the case of rock the quantity, quality, density, durability, and grading should be determined. Producer service records are helpful in selecting sources of construction materials.

2-8. Ice Conditions. Open-coast harbors built seaward from the shoreline and protected by massive breakwaters are seldom affected to any great extent by ice. Longshore currents or prevailing winds will cause ice transport, and the breakwater design should be such that this ice will not be trapped. If ice is trapped it should be easily flushed out by tides and currents. Breakwaters designed to withstand large waves are usually not damaged by ice, except walls, railings, lights, or other structures on top of the breakwater can be severely damaged when ice rides over the breakwater. Ice forces may be the controlling design load for breakwaters built in mild wave environments. The crushing strength of ice is about 400 pounds per square inch, and thrust per linear foot is about 58,000 pounds per foot of depth. Structures subject to impacts

from floating ice should be capable of resisting 10 to 12 tons per square foot on the area exposed to the greatest thickness of floating ice. Detailed procedures for quantifying ice loadings are contained in EM 1110-2-1612.

a. Ice Forces on Piles. Lightly loaded piles can be lifted when ice that is frozen to the pile is subject to vertical movement by tides and seiche. Long-period oscillations allow the sheet ice to freeze at the pile, and buoyancy forces acting on the entire sheet may lift the pile before the ice fails. The second half of the oscillation does not return the pile to its original position since it takes a higher force to drive the pile. Figure 2-2 shows a typical pile driven narrow-end down. A fiberglass, PVC, or plastic vertical-sided sleeve (as shown on the right side of the figure) provides a surface along which the ice can slip. The sleeve should extend below the ice level at lower low water levels. Floes of broken ice can subject piles to abrasion and impact damage.

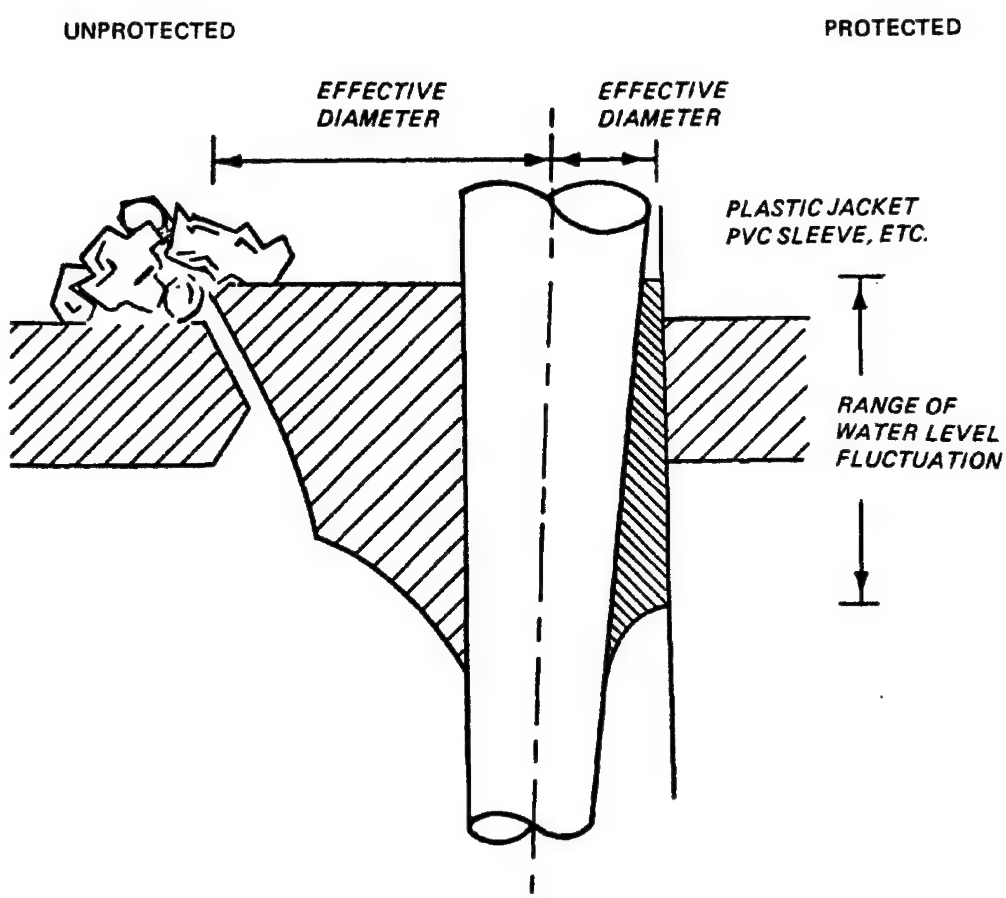


Figure 2-2. Typical pile showing protection and nonprotection from ice

b. Ice Forces on Rubble-Mound Structures. Smaller armor stone and concrete armor units are subject to lifting when ice that is frozen to them is moved vertically by tides and seiche. However, because of the original random orientation of these units, small vertical or horizontal position changes normally have no significant effect on stability. Individual armor units may also incur abrasive or impact damage from broken ice floes.

c. Ice Forces on Floating Breakwaters. Floating breakwaters are subject to the same lifting, abrasive, and impact forces described in a and b above. In many instances, floating structures are only used seasonally and are placed in a protective dry-dock during winter months if ice loadings are possible; however, evaluation of ice loadings merit careful attention since they may prove to be the controlling design loads.

2-9. Shoreline Changes.

a. General. Knowledge of the natural growth or the recession of the shoreline and of the offshore hydrography is needed to predict the impact of a project. If the project creates adverse impacts such as accretion or erosion, suitable mitigation measures such as sand bypassing or beach protection structures may be required.

b. Evaluation Methods. Historic changes can be determined from old charts or photographs. The NOS survey sheets are a good source of information since they show actual soundings of most coastal areas dating to the early 1800's. Care must be taken when comparing old survey data to assure that horizontal and vertical controls are corrected to a common reference. Old photographs can give approximate indications of changes; however, quantitative comparisons are difficult because water levels (tide, lake fluctuations, or river stages) are usually unknown. Surveys taken after completion of the project should always be made at the same time of the year to avoid inclusion of seasonal changes.

2-10. Prior Projects and Their Effects. Previous projects of similar type and scope often provide valuable information. While a new breakwater or jetty project is in the design stage a comprehensive review of similar projects may yield guidance to solutions of unanswered design questions. Most importantly, this review may stimulate consideration and analysis of problem areas that would have otherwise been overlooked.

2-11. Baseline Surveys. Physical and environmental surveys of the project site are needed during the preconstruction design phase. Bathymetric and hydraulic survey data are also to be used for model construction and verification. The following surveys are usually needed:

a. Bathymetric and topographic.

b. Beach profile.

c. Waves: Height, period, direction, and duration (spectral distribution of wave energy may be needed).

d. Currents: Velocity, direction, and duration.

e. Sediment: Suspended and bedload.

f. Beach composition.

g. Foundation conditions.

h. Wind: Speed, direction, and duration.

i. Ice: Frequency, duration, and thickness.

j. Biological population: Type, density distribution, and migration.

k. Water quality.

Dredged material water-disposal sites will usually need data from the a, d, j, and k baseline surveys.

2-12. Design Life, Degree of Protection, and Design Conditions. The project design life and the degree of protection are required before design conditions can be selected. The economic design life of most breakwaters and jetties is 50 years. The degree of protection during the 50-year period should be selected by an optimization process of frequency of damages (both to the structure and the area it protected) when waves exceed the design wave. Figure 2-3 and item 3 show the statistical relationship of project life, chance of event exceedance, and return period of event. Figure 2-3 shows that a wave with a height equal to or greater than the 100-year return period wave has a 39 percent chance of being exceeded during a 50-year project life. Chance of event exceedance may also be determined from figure 2-4. Design optimization is discussed in Chapter 10.

2-13. Dredging and Disposal. Dredging may be required to gain access to the site, for entrenching toe materials, or for various other reasons. When dredging is necessary a study should be conducted to determine volume of dredging, transport method, and the short- and long-term disposal impacts. Beneficial uses of dredged material should also be considered. Guidance on dredging disposal and beneficial uses of dredged material can be found in EM 1110-2-5025.

a. Dredges. The type of dredging equipment required should be suited to the wave environment and water depths characteristic of the project site. Rock or coral excavations normally require blasting with material removal by a clam shell shovel. Soft materials can be expediently handled with pipeline dredges.

PERCENT CHANCE OF GETTING ONE OR MORE SUCH OR BIGGER WAVES IN THIS MANY YEARS					WAVE RETURN PERIOD, YEARS
ONE HUNDRED YEARS	FIFTY YEARS	TWENTY-FIVE YEARS	TEN YEARS	ANY ONE YEAR	
				50	2
				40	
				30	
				25	
				20	5
		99	80	15	
	99.9	94	65	10	10
	90.5	71	40	5	20
86	61	40	18	2	50
64	39	22	9.6	1	100
40	22	12	5	0.5	200
18	9.5	5	2	0.2	500
10	4.8	2.5	1	0.1	1000
5	2.3	1.2	0.5	0.05	2000
2	1.0	0.5	0.2	0.02	5000
1	0.5	0.25	0.1	0.01	10,000

Figure 2-3. Relationship of project life, chance of event exceedance, and return period of event

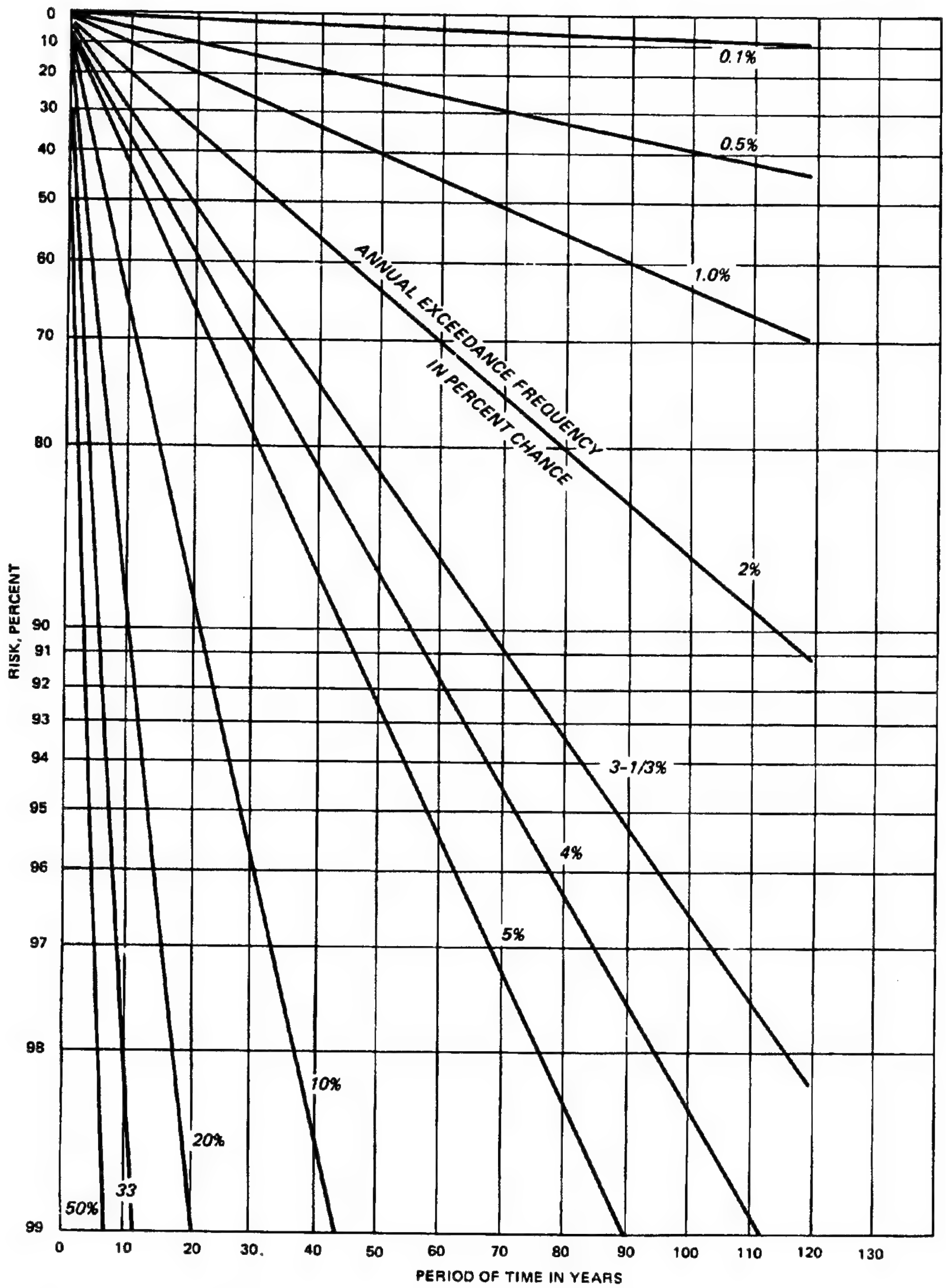


Figure 2-4. Risk of one or more events exceeding a given annual exceedance frequency within a period of time

b. Disposal Methods. Dredged material can be disposed of in open water or behind confinement dikes. Contaminated material is generally disposed of behind containment dikes, with careful monitoring of return water quality.

2-14. Seismic Design. Since failure of most breakwater and jetty projects as a result of an earthquake will not result in catastrophic consequences, these structures are generally not designed with seismic considerations. For projects located in high seismic risk zones, however, the geotechnical evaluation for these projects should at least consider the potential impact of seismic damage. If the cost to repair the seismic damage is considerable, as compared with the replacement cost, a detailed seismic evaluation may be warranted. The decision to design for seismic considerations should be decided on a case-by-case basis.

2-15. Environmental Impact. Environmental impacts generally fall into three categories: (a) dredging and disposal, (b) water quality impact of the project during normal operation, and (c) induced erosion or accretion. Both short-term construction and long-term impacts should be considered. Chapter 8 discusses environmental impacts.

2-16. Model Tests. Hydraulic model tests provide valuable input to breakwater and jetty design. Normally, proposed structure sections are optimized with a two-dimensional (2-d) stability model. The stability model results are used as input to final selection of structure details such as armor weight, crown height and width, and toe dimensions. The complexity of the breakwater head will determine whether three-dimensional (3-d), angular-wave attack stability tests are needed.

2-17. Operation and Maintenance (O&M). A comprehensive plan of how the project will be operated and maintained is required. This plan is presented in support of the operation and maintenance (O&M) costs. The following elements are normally included in the O&M plan.

a. Predicted Project Costs and Physical Changes. Include the post-construction prediction of physical changes and anticipated O&M costs.

b. Surveillance Plan. Describe the type and frequency of post-construction surveys. These could be hydrographic, aerial photos, beach profile, tide and wave records, and stability. The plan covers minimum monitoring of project performance to verify safety and efficiency. Cost information is for O&M budgetary purposes.

c. Analysis of Survey Data. Comparative studies of the survey data are required. These comparative studies verify design information such as rates of erosion, shoaling, and jetty deterioration.

d. Periodic Inspections and Project Performance Assessment. Present a tentative periodic inspection schedule. Inspections include a site assessment and a comparison of survey data with project changes predicted during the design effort.

CHAPTER 3

BREAKWATER AND JETTY PLANS

3-1. Objective. The layout of a breakwater or a jetty will depend on the intended function of the structure. A breakwater used to protect a small-boat harbor must reduce wave heights to an acceptable level in the interior channels and moorage area whereas a jetty used to stabilize an ocean inlet must reduce or eliminate channel shoaling. The goal of jetty placement is to direct tidal currents to keep the channel scoured to a suitable depth, much the same as the function of a river training dike.

3-2. Layout Options. Many options are available for breakwater and jetty layouts. The option selected must ensure that the structure functions as desired, is cost effective, and meets socio-economic constraints. Major layout options are presented below.

a. Attached or Detached. Jetties are usually attached to dry land in order to perform their function of stabilizing an inlet or eliminating channel shoaling. Breakwaters may be able to most economically serve their purpose either as attached or detached structures. If the harbor to be protected is on the open coastline and the predominant wave direction is such that wave crests approach parallel to the coastline, a detached offshore breakwater might be the best option. An attached breakwater extended from a natural headland could be used to protect a harbor located in a cove. As shown in figure 3-1 a system of attached and detached breakwaters may be used. An advantage of attached breakwaters is ease of access for construction, operation, and maintenance; however, one disadvantage may be a negative impact on water quality due to effects on natural circulation.

b. Overtopped or Nonovertopped. Overtopped structures are built to a crown elevation which allows larger waves to wash across the crest; therefore, wave heights on the protected side are larger than for a nonovertopped structure. Nonovertopped structures are constructed to an elevation that precludes any significant amount of wave energy from coming across the crest. Nonovertopped breakwater or jetty sections provide a greater degree of wave protection than overtopped structures, but they are more costly to build because of the increased volume of materials required. Selection of crest elevation, and thus amount of wave overtopping expected, can be optimized in a hydraulic model investigation by determining the magnitude of transmitted wave heights associated with various crest elevations, with the optimum crest elevation usually being the minimum structure height that provides the needed degree of wave protection. The crest elevation of an overtopped breakwater can sometimes be set by the design wave height that can be expected during the period the harbor will be used. This is especially true in colder climates. Overtopped structures are more difficult to design because their stability response is strongly affected by small changes in the still water level (swl).

c. Submerged. There may be instances where the needed degree of wave

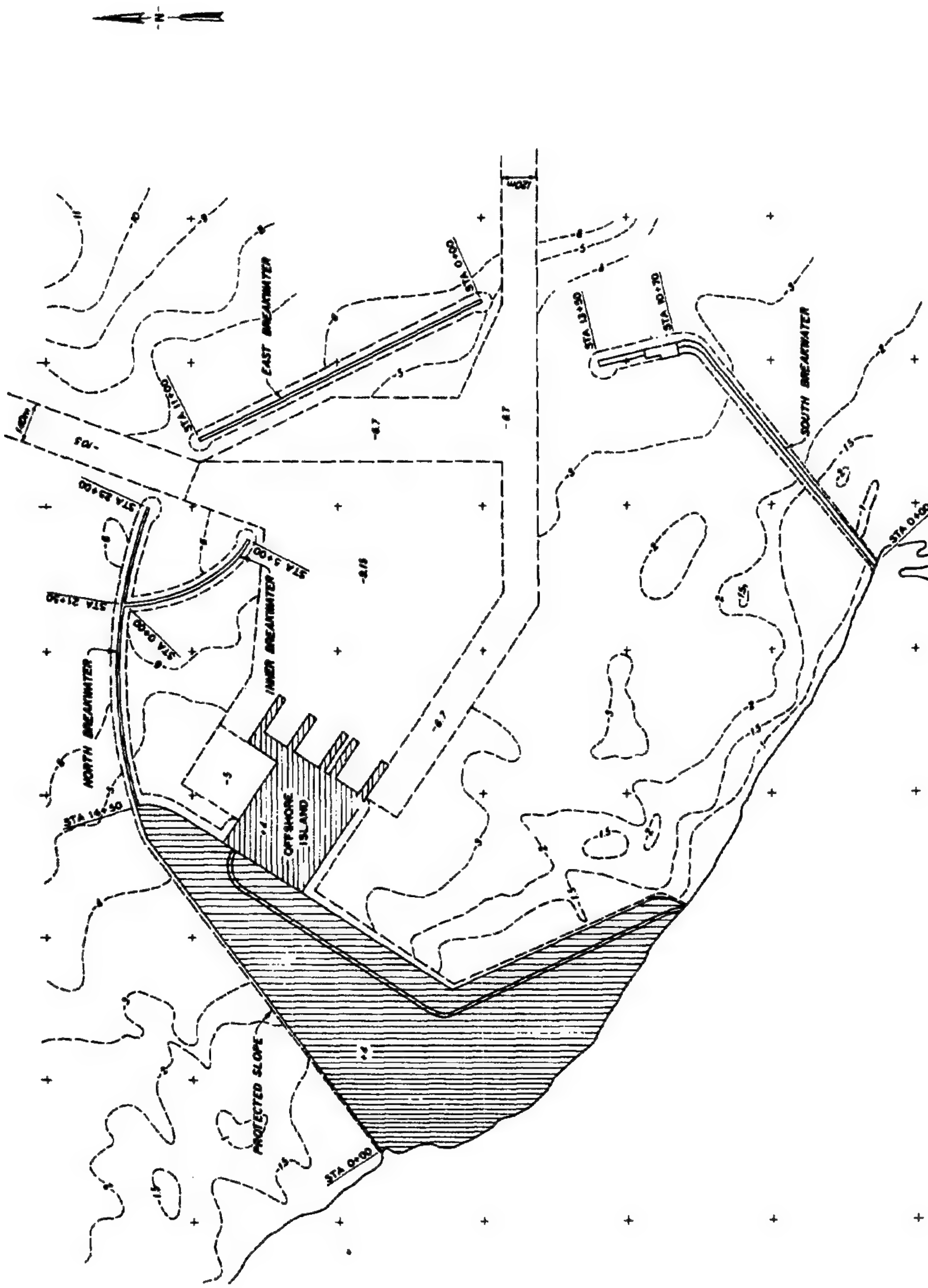


Figure 3-1. System of attached and detached breakwaters

protection can be achieved with submerged structures such as a detached breakwater constructed parallel to the coastline and designed to dissipate sufficient wave energy to eliminate or reduce shoreline erosion. Submerged breakwaters are less expensive to build than high-crested types and may be aesthetically more pleasing since they do not encroach on any scenic view which may be present. Some disadvantages, compared with a typical high-crested breakwater, are that significantly less wave protection is provided, monitoring the structure's condition is more difficult, and navigation hazards may be created.

d. Single or Double. Since the goal of jetty placement is to direct tidal currents to keep the channel scoured to a suitable depth, double parallel jetties will normally be required. However, there may be instances where coastline geometry is such that a single updrift jetty will provide a significant amount of stabilization. One disadvantage of single jetties is the tendency of the channel to migrate toward the structure. Choice of single or double breakwaters will depend on such factors as coastline geometry and predominant wave direction. Typically, a harbor positioned in a cove will be protected by double breakwaters extended seaward and arced toward each other with a navigation opening between the breakwater heads. For a harbor constructed on the open coastline a single offshore breakwater with appropriate navigation openings might be the more advantageous.

e. Weir Section. Some jetties are constructed with low shoreward ends that act as weirs. Water and sediment can be transported over this portion of the structure for part or all of a normal tidal cycle. The weir section, generally less than 500 feet long, acts as a breakwater and provides a semi-protected area for dredging of the deposition basin when it has filled. The basin is dredged to store some estimated quantity of sand moving into the basin during a given time period. A hydraulic dredge working in the semi-protected waters can bypass sand to the downdrift beach. Additional information on weir sections can be obtained from item 140. Figure 3-2 shows a typical weir section in a jetty system.

f. Deflector Vanes. In many instances where jetties are used to help maintain a navigation channel, currents will tend to propagate along the oceanside of the jetty and deposit their sediment load in the mouth of the channel. As shown in figure 3-3, deflector vanes can be incorporated into the jetty design to aid in turning the currents and thus help to keep the sediments away from the mouth of the channel. Position, length, and orientation of the vanes can be optimized in a model investigation. It should be noted that at the time this manual was prepared, the deflector vanes shown in figure 3-3 had been model tested but had not been used in the prototype.

g. Arrowhead Breakwaters. When a breakwater is constructed parallel to the coastline, as shown in figure 3-4, navigation conditions at the navigation opening may be enhanced by the addition of arrowhead breakwaters. Prototype experience with such structures however has shown them to be of questionable benefit in some cases.

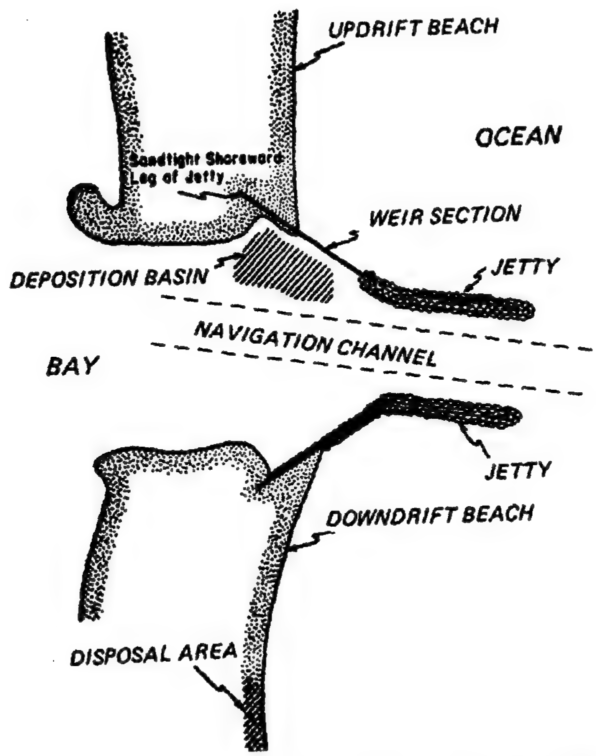


Figure 3-2. Key elements of a typical weir section in a jetty system

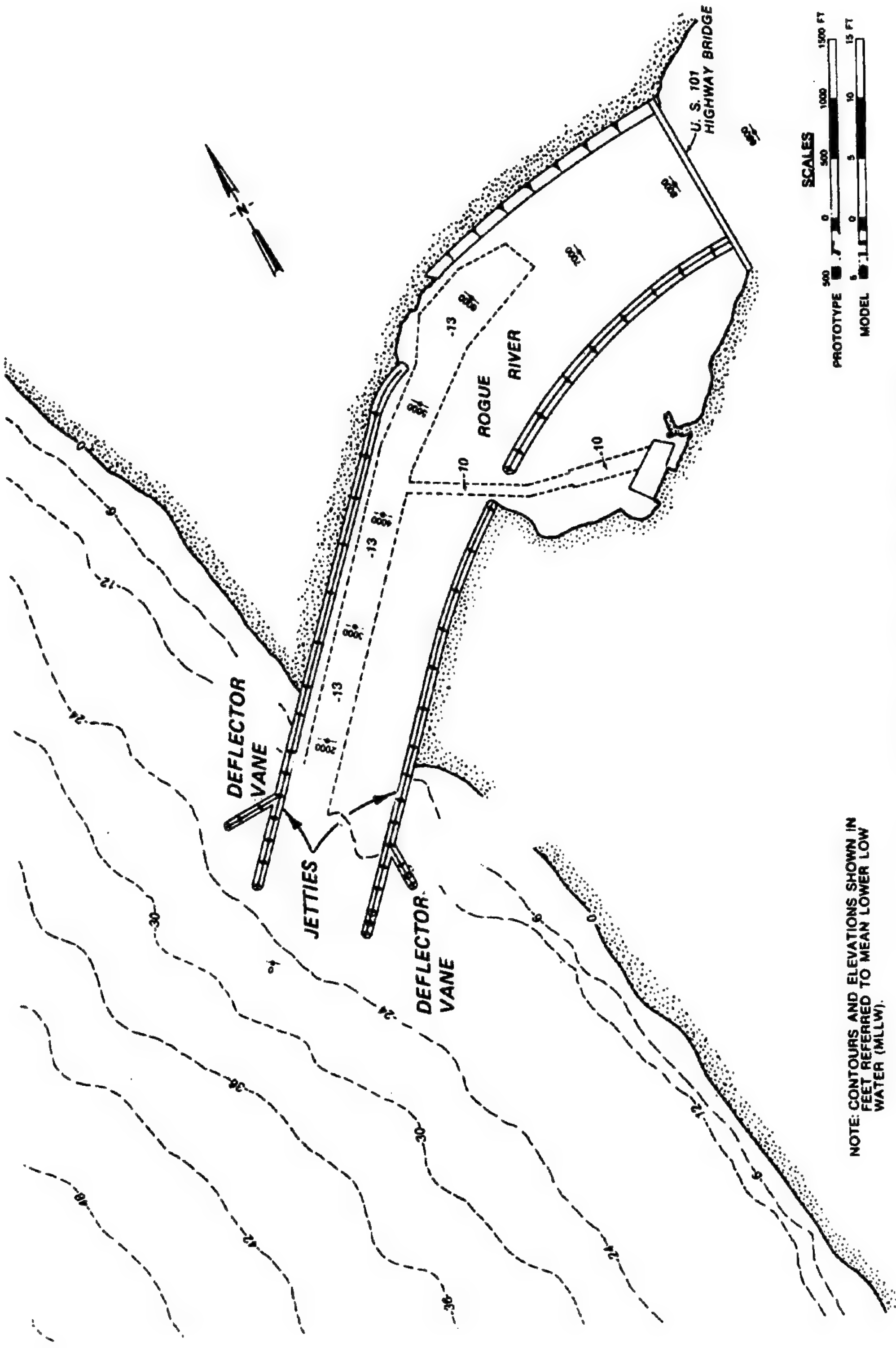


Figure 3-3. Typical use of deflector vanes

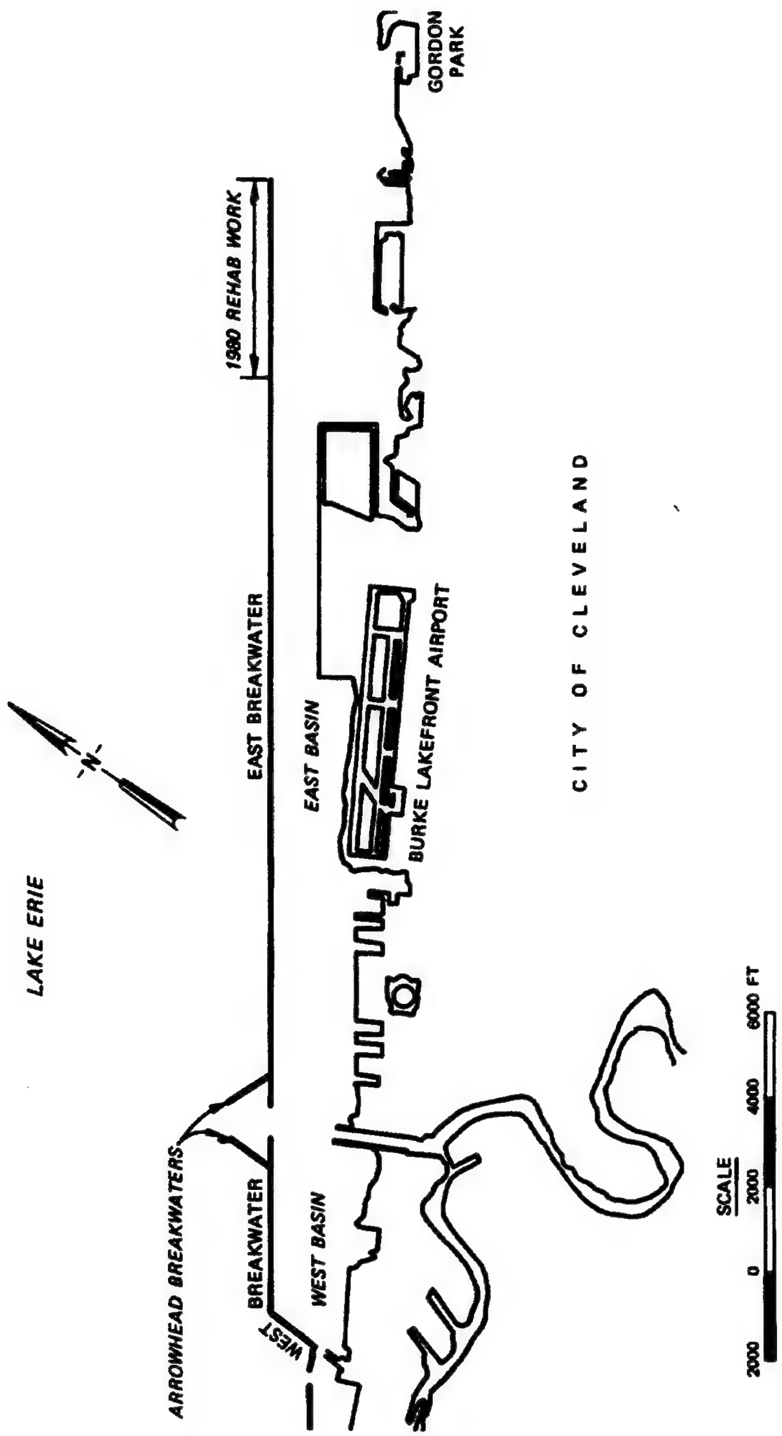


Figure 3-4. Use of arrowhead breakwaters at navigation opening

3-3. Selection of Structure Types. The type of structure selected should be the one that is the most economical, considering both the initial and annual maintenance costs. Also, it should be the one that is the most suitable under the conditions of exposure, depth of water, and nature of the foundation. Breakwaters may be classified as rubble-mound, vertical or wall type, floating, and other.

a. Rubble-mound Breakwaters. Rubble-mound breakwaters are adaptable to a wide range of water depths, suitable on nearly all foundations, readily repaired, and produce less reflected wave action than the wall type. However, they require larger amounts of material than most other types.

b. Wall-type Breakwaters. The wall type includes all structures in which the exposed faces are vertical or slightly inclined. Sheet-pile walls and sheet-pile cells of various shapes are in common use. Reflection of energy and scour at the toe of the structure are important considerations for all vertical structures. If forces permit and the foundation is suitable, steel-sheet pile structures may be used in depths up to about 40 feet. When foundation conditions are suitable, steel sheet piles may be used to form a cellular, gravity-type structure without penetration of the piles into the bottom material.

c. Floating Breakwaters. Floating breakwaters have potential application for boat basin protection, boat ramp protection, and shoreline erosion control. Conditions that favor floating breakwaters are as follows:

(1) Short-period waves. Dependent upon the type of floating structures, the maximum wave period for which the structures are effective ranges from 4 to 6 seconds. The sloping float breakwater (semi-submerged) provides protection intermediate to that achieved by floating breakwaters and fixed breakwaters, i.e., it may prove to be a desirable alternative for protection against 6- to 10-sec waves.

(2) Deep water. Water depth has little influence on in-place costs or performance.

(3) Fluctuating water levels. Where large tidal fluctuations or fluctuating reservoir pool elevations are encountered, the mooring line systems for floating structures can be adjusted to keep the breakwater in its optimum performance configuration.

(4) Water quality constraints. Interference with natural water circulation is minimal.

(5) Ice problems. If ice formation is anticipated, the structures can be towed to a protected area.

(6) Poor foundations. May be the only practical solution where foundation conditions will not support bottom-connected breakwaters.

(7) Aesthetics. Floating breakwaters have a low profile and present a minimum intrusion on the horizon, particularly for areas with large tidal ranges.

CHAPTER 4

DESIGN OF RUBBLE-MOUND STRUCTURES

4-1. Definition. A rubble-mound structure is composed of several layers of stone protected with a cover layer of selected armor stone or specially shaped concrete armor units. Armor used in the protective cover layer is usually placed in a random manner; however, under some circumstances stability can be improved by special placement techniques. A wide variety of cross-sectional shapes is possible.

4-2. Selection of Design Wave.

a. Flexible structures such as rubble mounds are usually designed for the significant wave height, H_s. In selected coastal areas such as the Pacific Northwest, in order to minimize repair costs, a less frequent (higher) design wave may be advisable (the average height of the highest 10 percent of all waves ($H_{1/10}$) has been used for the Pacific Northwest). Assuming a Rayleigh wave-height distribution, the designer may define H_s in approximate relation to other height parameters of the statistical wave-height distribution as follows:

Wave-Height Designation	Ratio of Wave-Height Designation to Significant Height
Average of all waves (H_{avg})	0.63
Average of highest one-third of all waves ($H_{1/3}$ or H_s)	1.00
Average of highest 10 percent of waves ($H_{1/10}$)	1.27
Average of highest 1 percent of all waves ($H_{1/100}$)	1.67
Expected maximum in 500 waves (H_{max})	1.86

b. Selection of a design wave height also depends on whether a structure will be subject to attack by depth-limited breaking waves. The depth-limited breaking wave should be calculated and compared with the unbroken storm wave height, and the lesser of the two chosen as the design wave.

c. Analysis of experimental data shows that the relationship between breaker height H_b and depth of breaking d_b is much more complex than indicated by the equation $H_b = 0.78d_b$. The dimensionless ratio d_b/H_b varies with nearshore slope m and wave steepness H/L . Breaking wave heights

typically vary from $0.60d_b$ to $1.1d_b$ over the range of conditions that rubble-mound structures are designed to withstand. Maximum depth-limited breaking wave heights can be estimated by following procedures described in Chapter 7 of the Shore Protection Manual (SPM) (item 132) or they can be determined by model study if site-specific conditions warrant.

d. Many structures, such as shore-connected breakwaters, are founded in variable water depths. Under these circumstances the structure can be designed in segments with smaller depth-limited design waves for the inshore areas. However, experience with concrete armor units, such as dolos, has shown that decreasing unit size inshore may not achieve any cost reduction due to increased forming and placing costs.

4-3. Concrete Armor Units. A multitude of concrete armor unit shapes have been developed over the past 30 years (figure 4-1). The major advantage of using concrete armor units is the increased stability of the structure while the primary disadvantage is the breakage of individual units. Concrete armor units have higher stability coefficients than stone units. Therefore, rubble structures can be built with steeper slopes and/or lighter weight armor units. Rubble-mound structures protected with concrete armor units deteriorate more rapidly than those armored with stone. Therefore, it may be prudent to conduct hydraulic model stability investigations for the final design so that risk of failure and anticipated maintenance can be adequately evaluated. Table 4-1 is a compilation of the types of concrete armor units that have been cited in technical literature. Several of the types listed therein have been used by the Corps of Engineers. Major areas of consideration for evaluating potential use of concrete armor units are summarized as follows:

a. Availability of Casting Forms.

(1) Forms for manufacturing concrete armor units can be obtained from District offices, private industry with forms in stock, and private companies that build forms. Forms should be designed to compensate for concrete shrinkage so that excessive internal stresses are not created in the trunk section of the armor unit.

(2) The only District office that currently has forms available is the Philadelphia District. They have eight 16-ton dolos forms which are available for loan to other Corps field offices at no cost. Contact is NAPEN-N.

b. Concrete Quality. Concrete performance in the marine environment depends primarily on concrete quality. Procedures for the investigation and selection of appropriate concrete materials are given in EM 1110-2-2000. The concrete should have low permeability, a water-cement ratio suited to the exposure conditions, adequate strength, proper air-entrainment, durable aggregates, and adequate cover over reinforcing steel. Normal weight aggregate

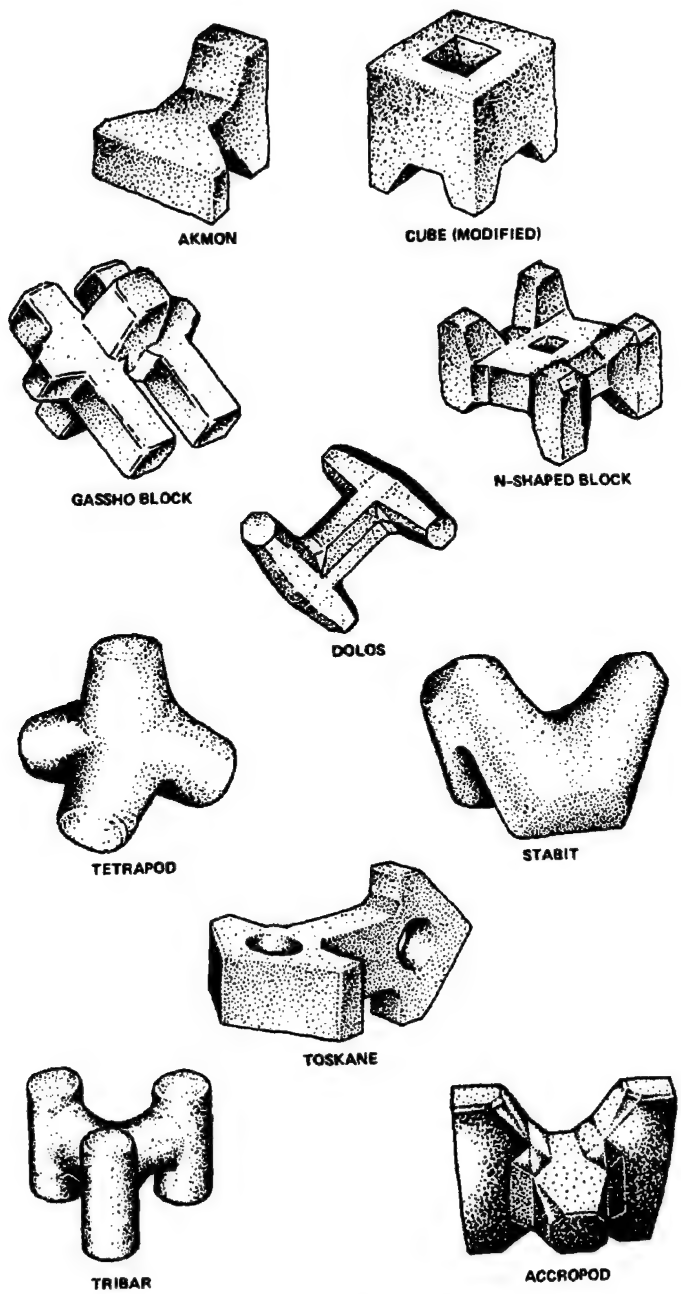


Figure 4-1. Concrete armor unit shapes

Table 4-1. Types of Concrete Armor Units

Name of Unit	Development of Unit: Country	Development of Unit: Date	Bibliography Item No.
Akmon	Netherlands	1962	105
Binnie Block	England	--	105
Bipod	Netherlands	1962	105
Cob	England	1969	63
Cube(1)		(2)	63
Cube (Modified)(1)	United States	1959	76
Dolos(1)	South Africa	1963	93
Dom	Mexico	1970	--
Gassho Block	Japan	1967	98
Grobbelar	South Africa	1957	131
Hexaleg Block	Japan	--	50
Hexapod(1)	United States	1959	76
Hollow Square	Japan	1960	98
Hollow Tetrahedron	Japan	1959	98
Interlocking H-Block	United States	1958	132
Mexapod	Mexico	1978	108
N-Shaped Block	Japan	1960	98
Pelican Stool(1)	United States	1960	70
Quadripod(1)	United States	1959	76
Rectangular Block(1)	--	(2)	76
Rentrapod	--	--	--
Seabee	Australia	1978	15
Shed	England	1982	37
Stabilopod	Romania	1965	--
Stabit	England	1961	120
Sta-Bar(1)	United States	1966	104
Sta-Pod(1)	United States	1966	104
Stolk Cube	Netherlands	1965	55
Svee Block	Norway	1961	126
Tetrahedron (Solid)(1)	--	(3)	76
Tetrahedron (Perforated)(1)	United States	1959	76
Tetrapod	France	1950	43
Toskane(1)	South Africa	1966	131
Tribar	United States	1958	106
Trigon	United States	1962	--
Tri-Long	United States	1968	--
Tripod	Netherlands	1962	105
Tripod Block	England	1974	12

(1) The units have been tested, some extensively, at WES.

(2) Cubes and rectangular blocks are known to have been used in masonry-type breakwaters since early Roman times and in rubble-mound breakwaters during the last centuries. The cube was tested at WES as early as 1943.

(3) Solid tetrahedrons are known to have been used in hydraulic works for many years. This unit was tested at WES in 1959.

concrete has a unit weight that ranges from 140 to 155 pounds per cubic foot. The minimum stable weight of an individual armor unit is inversely proportional to its density cubed; therefore, every effort should be made to maximize the density of the concrete. A minimum density of saturated concrete should be stipulated.

c. Use of Reinforcing. No firm guidance is available on how much and what type of reinforcement should be used in concrete armor units. General guidelines indicate that units weighing 20 tons or more, which are placed by land-based equipment, may require reinforcing. If a floating plant is used for placements, then units weighing 10 tons or more may require reinforcing. There are various opinions on what type of reinforcing should be used. A conventional reinforcing-bar cage has been used and although it provides dowling of the unit, should breakage occur, it is questionable whether rebar in the cage can be placed close enough to the surface of the concrete to provide adequate strength without being exposed to possible long-term corrosion caused by concrete surface cracking. Fiber reinforced concrete (FRC) which improves the first crack strength and impact resistance of concrete has high potential for use in concrete armor units, but at this time, no significant data exist on how FRC armor units will perform. A limited number of experimental fiber reinforced dolosse were placed on the North Jetty at Humboldt Bay, California (item 62), and at Cleveland Harbor, Ohio, but their exposure to wave action has not been sufficient to form definite conclusions. Fiber reinforced dolosse are proposed to be included in the 1986 repair of the Crescent City Harbor Breakwaters and Humboldt Bay Jetties and should provide meaningful field experience. Another guide for deciding if reinforcing is required is to determine if the units rock back and forth or if they are displaced under attack of design wave conditions during the hydraulic model investigation; if either significant amounts of rocking or displacement are observed for the selected design conditions, then reinforcing should be used. Hydraulic studies have indicated that up to 15 percent random breakage of dolos armor units may be experienced before stability is threatened, and up to five broken units in a cluster can be tolerated. An evaluation of the consequences and replacement cost of broken concrete armor units must be compared with the cost of reinforcing all the concrete armor units or those in selected portions of the structure, such as the head or sections where wave energy will be focused.

d. Armor Unit Placement. Placement of toe units is critical to the overall stability of the structure. A toe trench or berm of apron material should be constructed to provide bracing up to at least one-half the height of the toe units. Site-specific model studies (items 8, 22, 33, and 87) have shown that when dolosse are used, turning the vertical leg away from the slope, as shown in figure 4-2, provides improved stability. In some instances on-slope placement of concrete armor units in a specified pattern may provide greater stability than random placement (for dolosse see Pattern No. 3, item 23). This, plus the added ease and reliability of monitoring armor unit movement, may justify specifying pattern placement for some structures. If crown access is required some type of cap will probably be needed. Ribbed

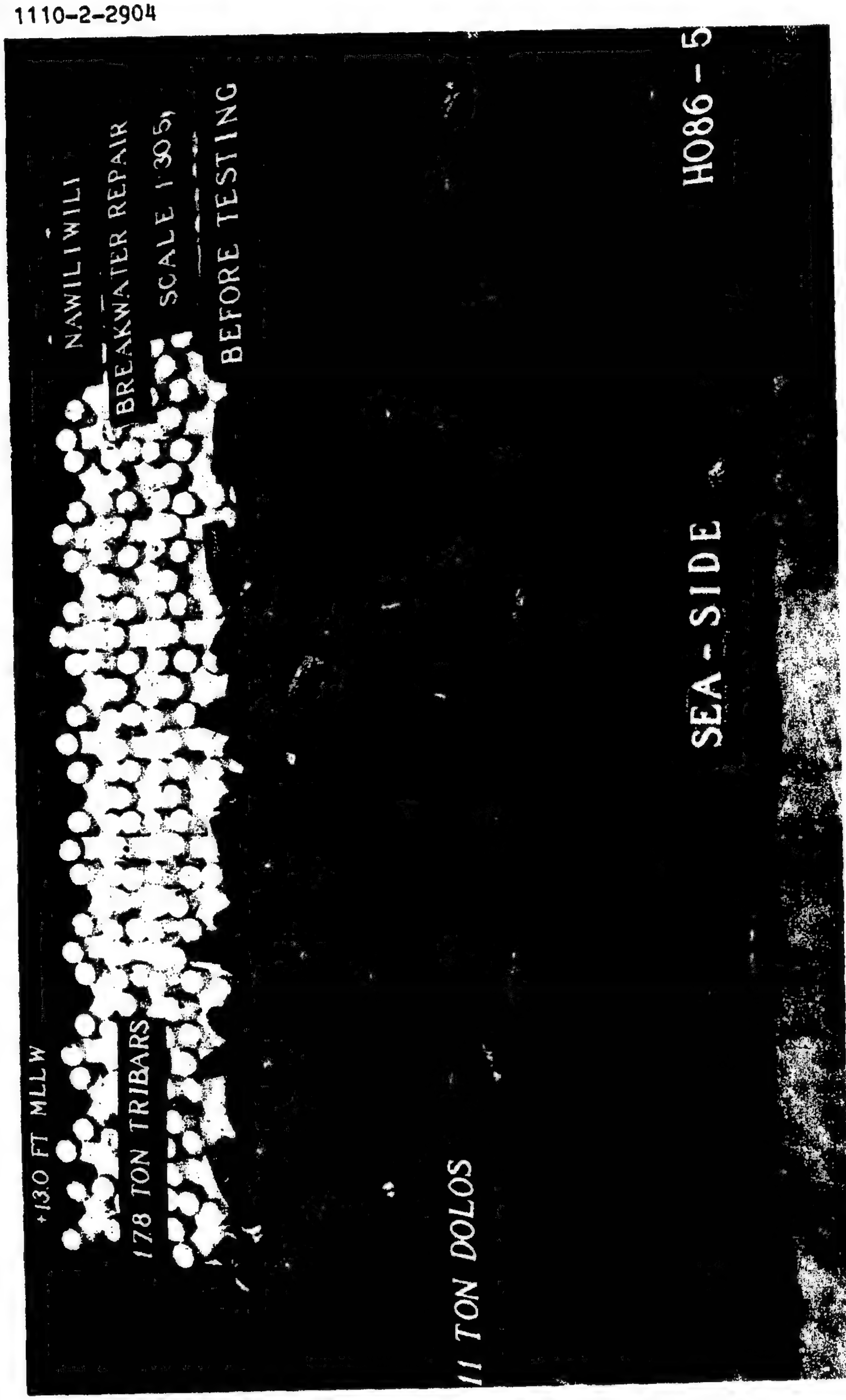


Figure 4-2. Special placement of dolos toe units

caps, shown in figure 4-3, used extensively in the Hawaiian Islands, have been found to withstand the wave environment better than solid monolithic caps. The porosity of the caps also appears to improve seaward armor stability. One disadvantage of ribbed caps is increased wave transmission.

4-4. Special Stone Placement. Placed stone construction has been used successfully on the Great Lakes and Oregon coasts. This method requires that stone be placed with the long axis normal to the slope. Site-specific model tests of the south jetty at Tillamook Bay, Oregon (item 91), showed that the stability coefficient for placed stone can be as high as 22. Use of the placed stone technique requires careful attention to construction detail. The following description of placed stone construction should be used in armor stone specifications:

> Each stone will be individually placed by equipment suitable for lifting, manipulating, and placing stones of the size and shape specified. No stone shall have a longest dimension less than two nor more than three times its shortest dimension as determined along perpendicular axes passing through the approximate center of gravity. Each stone shall be placed with its longest axis perpendicular to the armor slope. Placing efforts shall insure that each stone is firmly set and supported by underlying materials and adjacent stones. Loose stones shall be reset or replaced.

4-5. Overtopped Breakwaters. Traditional high-crested breakwaters with a multilayered cross section may not be appropriate for a structure used to protect a beach or shoreline. Adequate wave protection may be more economically provided by a low-crested or submerged structure composed of a homogeneous pile of stone. Presently, a comprehensive investigation of this type of structure is being undertaken (item 2) and detailed design information should be available in the near future. Based on a preliminary analysis of these data, a stability coefficient of 4.0 may be used to size the stone for this type of structure.

4-6. Estimating Wave Runup. Wave runup is used as an aid in setting crest elevation, determining constructability, designing backslope armor, and estimating transmitted wave heights. Preliminary design can use the methods presented in the SPM (item 132) for estimating runup.

4-7. Selection of Armor Type and Weight.

a. Many design requirements are most advantageously met by stone armor; however, some design wave conditions may be of such magnitude that the protective cover layer must consist of specially shaped concrete armor units in order to provide economical construction of a stable breakwater. Choice of stone or concrete armor units will depend primarily on design wave conditions,

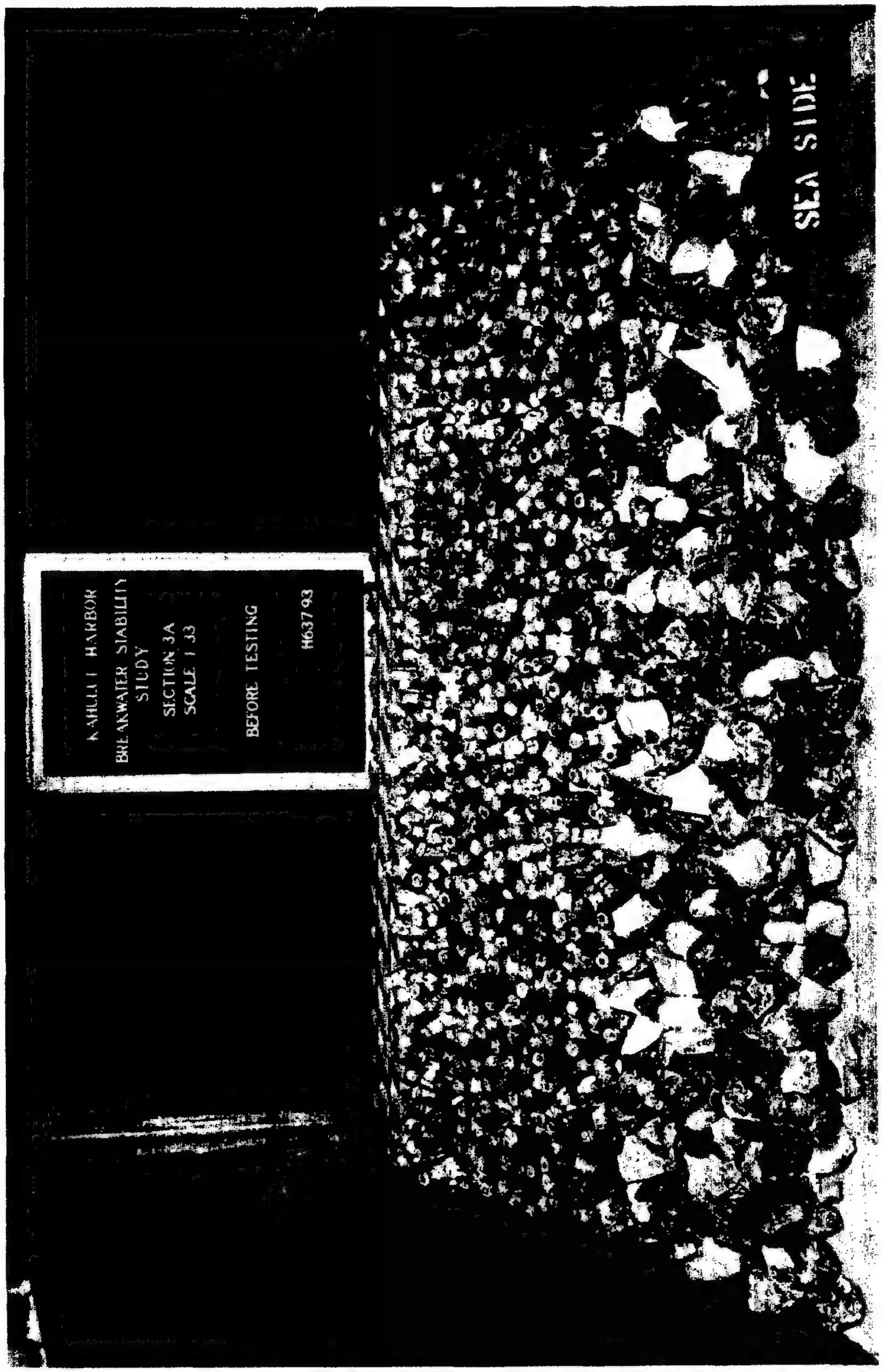


Figure 4-3. Use of concrete ribbed cap

availability of materials, and capabilities of available construction equipment.

b. Numerous hydraulic model investigations have been conducted at WES in an effort to develop generalized design guidance for rubble-mound structures. These studies yielded information on stone (items 26, 27, and 62) tribars and tetrapods (items 66 and 76), quadripods (items 69 and 76), modified cubes, hexapods, and modified tetrahedrons (item 76), dolosse (item 27), and toskanes (item 24).

c. Results of stability tests described in the above-mentioned investigations are reasonably well correlated by the Hudson stability equation; i.e.,

$$W_a = \frac{\gamma_a H^3}{K_D (S_a - 1)^3 \cot \alpha} \qquad (4\text{-}1)$$

where

W_a = weight of an individual armor unit, pounds

γ_a = unit weight of armor unit, pounds per cubic foot

H = design wave height, feet

K_D = stability coefficient, dimensionless

S_a = specific gravity of armor unit, relative to water in which the breakwater is constructed $(S_a = \gamma_a/\gamma_w)$

α = angle of structure slope measured from horizontal, degrees

d. The following restrictions should be observed when using the Hudson stability equation to estimate required armor unit weights:

(1) Values of K_D should not exceed those shown in table 4-2.

(2) The equation is intended for a structure with a crest high enough to prevent major wave overtopping.

(3) The equation is valid only for armor units of nearly uniform size. For stone, the size range should be restricted within 0.75W to 1.25W.

Table 4-2. Suggested K_D Values for Use in Determining Armor Unit Weight.
(No-Damage Criteria and Minor Overtopping)

			Structure Trunk K_D[b]		Structure Head K_D		Slope
Armor Units	n[a]	Placement	Breaking Wave	Nonbreaking Wave	Breaking Wave	Nonbreaking Wave	Cot α
Quarrystone							
Smooth rounded	2	Random	1.2	2.4	1.2	1.9	1.5 to 3.0
Smooth rounded	>3	Random	1.6	3.2	1.4	2.3	(c)
Rough angular	1	Random[d]	(d)	2.9	(d)	2.3	(c)
					1.9	3.2	1.5
Rough angular	2	Random	2.0	4.0	1.6	2.8	2.0
					1.3	2.3	3.0
Rough angular	>3	Random	2.2	4.5	2.1	4.2	(c)
Rough angular	2	Special[e]	5.8	7.0	5.3	6.4	(c)
Parallelepiped[f]	2	Special[e]	7.0-20.0	8.5-24.0	--	--	(c)
Tetrapod					5.0	6.0	1.5
and	2	Random	7.0	8.0	4.5	5.5	2.0
Quadripod					3.5	4.0	3.0
					8.3	9.0	1.5
Tribar	2	Random	9.0	10.0	7.8	8.5	2.0
					6.0	6.5	3.0
Dolos	2	Random	15.0[g]	31.0[g]	8.0	16.0	2.0[h]
					7.0	14.0	3.0
Modified Cube	2	Random	6.5	7.5	--	5.0	(c)
Hexapod	2	Random	8.0	9.5	5.0	7.0	(c)
Toskanes	2	Random	11.0	22.0	--	--	(c)
Tribar	1	Uniform	12.0	15.0	7.5	9.5	(c)
Quarrystone (K_{RR}) Graded angular	-	Random	2.2	2.5	--	--	--

(a) n is the number of units comprising the thickness of the armor layer.

(b) Applicable to slopes ranging from 1 on 1.5 to 1 on 5.

(c) Until more information is available on the variation of K_D value with slope, the use of K_D should be limited to slopes ranging from 1 on 1.5 to 1 on 3. Some armor units tested on a structure head indicate a K_D slope dependence.

(d) The use of a single layer of quarrystone armor units subject to breaking waves is not recommended, and only under special conditions for nonbreaking waves. When it is used, the stone should be carefully placed.

(e) Special placement with long axis of stone placed perpendicular to structure face.

(f) Long slablike stone with the long dimension about three times its shortest dimension.

(g) Refers to no-damage criteria (<5 percent displacement, rocking, etc.); if no rocking (<2 percent) is desired, reduce K_D 50 percent.

(h) Stability of dolosse on slopes steeper than 1 on 2 should be substantiated by site-specific model tests.

NOTE: Breaking wave stability coefficients for stone and dolos were developed using a 1V:10H foreslope.

(4) Armor slope should be uniform and within the range of 1V:1.5H to 1V:3H.

(5) Specific weight of the armor units should be greater than 120 pounds per cubic foot and less than 180 pounds per cubic foot.

The required armor weight can be estimated using figure 4-4.

4-8. Selection of Seaside Armor Slope. Since the size of armor unit and wave runup increases as the slope becomes steeper, and volume of material required for construction increases as the slope becomes flatter, rigorous optimization requires an iterative scheme. However, practical considerations normally require the range of slopes to be between 1V:1.5H and 1V:2H. Steeper slopes are subject to landslide-type failures, and flatter slopes become prohibitively costly.

4-9. Selection of Harbor-Side Armor Slope. Comprehensive tests to determine the optimum harbor-side slope have not, as yet, been conducted. It is common practice to use slopes from 1V:1.25H to 1V:1.5H . The angle of repose for dumped stones is about 1V:1.25H . Thus, this slope is used when the structure is constructed by the end-dump method and there is only moderate wave action and minor overtopping. When the structure is designed for large waves and moderate overtopping, a harbor-side slope of from 1V:1.33H to 1V:1.5H is usually used. For large amounts of overtopping the steepest rear slope that will be stable is preferred.

4-10. Detailing Structure Cross Section.

a. General. A rubble structure is normally composed of a bedding layer and a core of quarry-run stone covered by one or more layers of larger stone and an exterior covering of large stone or concrete armor units. Typical rubble-mound cross sections for nonbreaking and breaking waves are shown in figure 4-5.

b. Crest Elevation and Width.

(1) Usually overtopping of a rubble structure such as a breakwater or jetty can be tolerated only if it does not cause damaging waves behind the structure. Whether overtopping will occur depends on the height of the crest of the structure relative to the height of wave runup. Wave runup depends on wave characteristics, design water level, structure slope, porosity, and roughness of the cover layer. The selected crest elevation should be the lowest that provides the protection required. Excessive overtopping of a breakwater or jetty can cause choppiness of the water surface behind the structure, and thus be detrimental to harbor operations. Operations such as mooring of small craft and some types of commercial unloading require calm waters. Overtopping of jetties may be tolerated if it does not adversely affect the channel.

γ = Pounds per cubic foot

Cot α	150	152	154	156	158	160	162	164	166	168	170	172	174	176	178	180
1.5	100	95	90	85	81	77	73	70	67	64	61	58	55	53	51	49
2.0	75	71	67	64	61	58	55	52	50	47	45	43	41	40	38	37
2.5	60	57	54	51	48	46	44	42	40	38	36	35	33	32	31	29
3.0	50	47	45	42	40	38	37	35	33	31	30	29	28	27	26	25
3.5	43	40	38	36	34	33	31	30	28	27	26	25	24	23	22	21

Percentage of W_{150}

Notes: Design-factor relationships are from the Waterways Experiment Station formula $W = \frac{\gamma H^3}{K_\Delta (S-1)^3 \cot \alpha}$ where Cot α = 1.5,

γ = 150 lb/cu ft, unit weight of water = 64 lb/cu ft. For fresh water, unit weight = 62.4; a factor of 0.88 must also be applied.

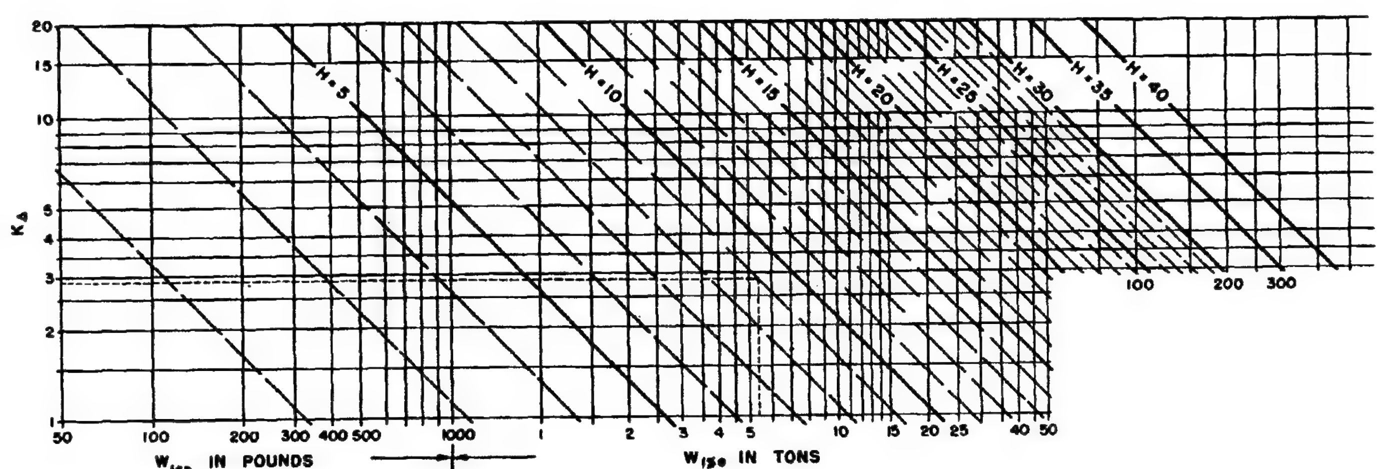
Subscript 150 indicates γ value of 150 lb/cu ft.

Problem: For the purpose of illustration the following values have been assumed: H = 9' K_Δ = 2.9 γ = 160 and Cot α = 2. The armor weight is desired.

Solution: Enter graph from left side at K_Δ = 2.9, follow horizontal dashed line to point of intersection with diagonal line H = 9, drop vertically, read W_{150} = 5.3 tons.

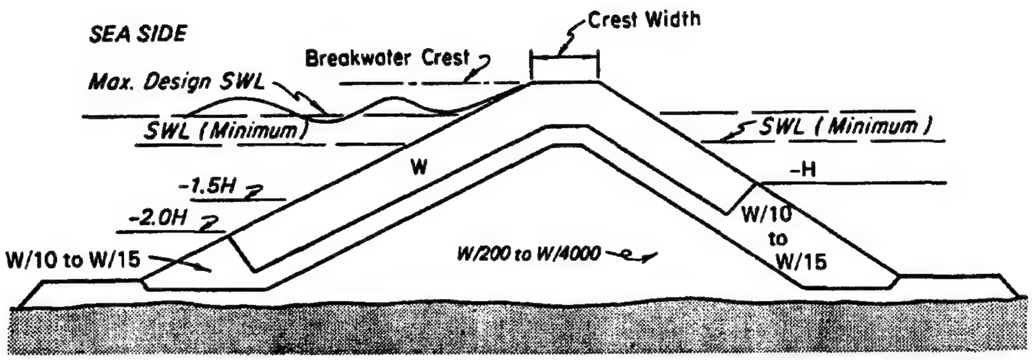
The value of 58 percent is obtained from the upper right-hand table for Cot α = 2 and γ = 160.

5.3 tons x 0.58 = 3.1 tons. If in fresh water 0.88 x 3.1 = 2.7 tons.

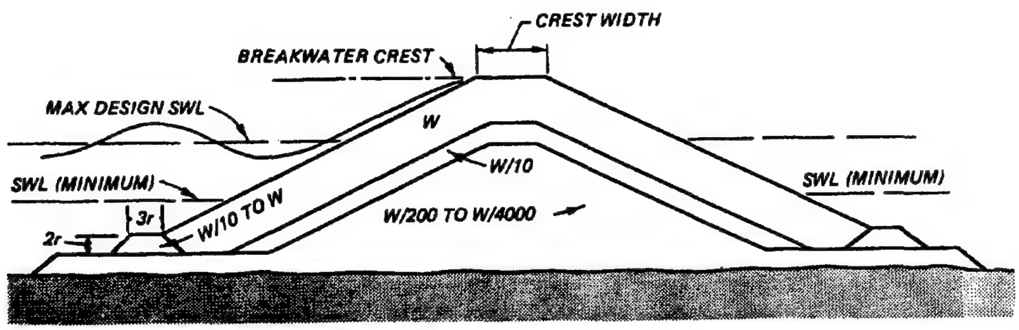


This method devised by R.Q. Palmer, Honolulu District

Figure 4-4. Armor weight estimations



a. RUBBLE-MOUND SECTION FOR SEAWARD WAVE EXPOSURE WITH ZERO-TO-MODERATE OVERTOPPING CONDITIONS



b. RUBBLE-MOUND SECTION FOR WAVE EXPOSURE FROM BOTH SIDES WITH MODERATE OVERTOPPING

ROCK SIZE	LAYER	ROCK SIZE RANGE (%)
W	PRIMARY COVER LAYER[1]	125 TO 75
W/10	TOE BERM AND FIRST UNDERLAYER[2]	130 TO 70
W/200	SECOND UNDERLAYER	150 TO 50
W/4000	CORE AND BEDDING LAYER	170 TO 30

H = WAVE HEIGHT
W = WEIGHT OF INDIVIDUAL ARMOR UNIT
r = AVERAGE THICKNESS OF ONE LAYER OF MATERIAL (n = 1)

Figure 4-5. Typical rubble-mound cross sections for nonbreaking and breaking waves

(2) Little study has been made of the effect crest width has on the stability of rubble-mound structures. As a general guide, minimum crest width should equal the combined widths of three armor units (assuming this width provides safe operation of construction and maintenance equipment operated from the structure). Minimum crest width may be obtained from the following equation.

$$\beta = 3k_{\Delta} \left[\frac{W_a}{\gamma_a} \right]^{1/3} \tag{4-2}$$

where

β = crest width, feet

k_{Δ} = layer thickness coefficient, dimensionless

W_a = weight of an individual armor unit, pounds

γ_a = unit weight of armor unit, pounds per cubic foot

Values of k_{Δ} are presented in table 4-3.

Table 4-3. Layer Thickness Coefficients and Porosities

Type of Armor Unit	$n^{(1)}$	Placing Technique	Layer Thickness Coefficient, k_{Δ}	Porosity Percent
Smooth stone	2	Random	1.00	38
Rough stone	2	Random	1.00	37
Tetrapod	2	Random	1.04	50
Quadripod	2	Random	0.95	49
Hexapod	2	Random	1.15	47
Modified Cube	2	Random	1.10	47
Tribar	2	Random	1.02	54
Tribar	1	Uniform	1.13	47
Toskane	2	Random	1.03	52
Dolos	2	Random	0.94	56

(1) Number of layers of armor units.

c. Thickness of Primary Armor Layer and Underlayer and Number of Armor Units. The thickness r of the cover and underlayers can be obtained from the following equation:

$$r = nk_{\Delta} \left[\frac{W_a}{\gamma_a}\right]^{1/3} \qquad (4\text{-}3)$$

where n equals number of layers. The number of armor units N_a required for a given surface area A can be determined by the following equation:

$$N_a = Ank_{\Delta} \left[1 - \frac{P}{100}\right] \left[\frac{\gamma_a}{W_a}\right]^{2/3} \qquad (4\text{-}4)$$

where P equals average porosity of a rubble structure cover layer.

d. Bottom Elevation of Armor Layer. Armor units in the cover layer should be extended downslope to an elevation below minimum still water level equal to 1.5H when the structure is in a depth greater than 1.5H. If the structure is in a depth of less than 1.5H, armor units should be extended to the bottom. Toe conditions at the interface of the breakwater slope and sea bottom are a critical stability area and should be thoroughly evaluated in the design.

e. Secondary Cover Layer.

(1) The weight of armor units in the secondary cover layer, between -1.5H and -2.0H, should be approximately equal to one-half the weight of armor units in the primary cover layer. Below -2.0H, the weight requirements can be reduced to approximately W/15 . When the structure is located in shallow water, where the waves break, armor units in the primary cover layer should be extended down the entire slope.

(2) The above-mentioned ratios between the weights of armor units in the primary and secondary cover layers are applicable only when stone units are used in the entire cover layer for the same slope. When precast concrete units are used in the primary cover layer, the weight of stone in the other layers should be based on the equivalent weight of stone armor. For example, tetrapods designed for nonbreaking wave attack on a structure trunk have a stability coefficient equal to 8.0 as opposed to 4.0 for rough angular stone. If the tetrapods have a unit weight of 150 pounds per cubic foot, are placed on a 1V:2H slope, and are designed for 20-foot nonbreaking waves, the required weight, as determined from equation 4-1, would be equal to 15.5 tons. If

stone armor with a unit weight of 165 pounds per cubic foot is to be used for the same conditions, the equivalent stone weight would be 21 tons. The secondary cover layer from -1.5H to the bottom should be as thick as or thicker than the primary cover layer.

f. Underlayers. The first underlayer (directly beneath the primary armor units) should have a minimum thickness of two stones (n = 2), and these should weigh about one-tenth the weight of the overlying armor units (W/10). This applies where the cover layer and first underlayer are both stone, and where the first underlayer is stone and the cover layer is concrete armor units with a stability coefficient $K_D \leq 10$. When the cover layer is of armor units with $K_D \geq 10$, the first underlayer stone should weigh about W/5 or one-fifth the weight of the overlying armor units. If a second underlayer is used it should have a minimum thickness of two stones; these should weigh about one-twentieth the weight of the overlying stones.

g. Bedding Layer or Filter Blanket.

(1) A rubble structure may be protected from excessive settlement resulting from leaching, undermining, or scour by the use of either a bedding layer or filter blanket. Filter fabric may be used as a substitute for a bedding layer or filter blanket to protect the foundation materials. When a fabric is used, a protective layer of spalls or crushed rock (7-inch maximum to 4-inch minimum size) having a recommended minimum thickness of 2 feet should be placed between the fabric and adjacent stone to prevent puncture of the fabric. Filter criteria should be met between the protective layer of spalls and adjacent stone.

(2) It is advisable to use a bedding layer or filter blanket to protect the foundations of rubble-mound structures from undermining except where (a) depths are greater than appromimately three times the maximum wave height, (b) the anticipated current velocities are too weak to move the average size of foundation material, or (c) the foundation is a hard, durable material (such as bedrock).

(3) When the foundation is a cohesive material a filter blanket may not be required. However, a layer of quarry spalls or other crushed stone or gravel may be placed as a bedding layer or apron to reduce scour of the bottom or settlement of the structure. Foundations of coarse gravel may not require a filter blanket. When the rubble structure is founded on sand, a filter blanket should be provided to prevent waves and currents from removing sand through the voids of the rubble and thus causing settlement.

(4) When large quarrystone are placed directly on a sand foundation at depths where waves and currents act on the bottom (as in the surf zone), the rubble will settle into the sand until it reaches the depth below which the sand will not be disturbed by the currents. Large amounts of rubble may be required to allow for the loss of rubble because of settlement. This, in turn, can provide a stable foundation.

(5) Gradation requirements of the bedding layer or filter blanket depend principally on the size characteristic of the foundation material. The criterion for filter design is D_{15} (filter) $\leq 5\ D_{85}$ (foundation); i.e., the diameter exceeded by the coarsest 85 percent of the filter material must be less than or equal to five times the diameter exceeded by the coarsest 15 percent of the foundation material. Quarry spalls, ranging in size from 1 to 50 pounds, will generally suffice if the bedding layer is placed on a filter cloth or a coarse gravel (or crushed stone) filter layer which meets the stated filter design criteria. Layer thickness depends generally on the depth of water in which the material is to be placed and the size of quarry-stone used, but should not be less than 2 feet to ensure that bottom irregularities are completely covered. It is common practice to extend the bedding layer at least 5 feet beyond the toe of the cover stone.

(6) Stability against wave attack of the exposed bedding material has been found to be analogous to the stability of the armor layer of a rubble-mound structure, with the exceptions that the slope of the seaward face, α, vanishes from the problem and the wavelength is considered (item 57). The required 50 percent weight (W_{50}) can be calculated from the following equation

$$W_{50} = \frac{\gamma_{50}\ H^3}{1.34 \times 10^5\ (S_{50} - 1)^3\ (1/L)^2} \tag{4-5}$$

where L is the local wavelength.

4-11. Use of Concrete Caps. Concrete caps may be considered for strengthening the crest, increasing crest height, or providing access along the crest for construction and maintenance purposes. Concrete caps used in conjunction with precast armor units provide a rigid backup to the top row of units. To evaluate the merits of using a concrete cap for increasing stability under overtopping conditions, consideration should be given to the cost of including a cap versus the cost of increasing breakwater dimensions. Structure settlement should be evaluated as it may cause the cap to be overstressed. Maintenance costs for an adequately designed rubble structure will probably be lower than any composite-type structure. Use of a concrete cap or crownwall has a significant influence on overall stability of the structure. In particular, the effects of increased back pressures should be considered.

4-12. Design of Structure Head and Lee-Side Armor.

a. Structure heads, normally exposed to a multiplicity of wave directions, present special design problems. Geometry of the head should be kept as simple as is functionally possible, and changes in structure slope should be accomplished by gentle transitions. Armoring of the head should be the same

on the lee-side slope as on the seaside slope for a distance of about 100 to 200 feet from the structure end. This distance depends on such factors as armor slope, crest elevation at the seaward end, and alignment with respect to direction of wave approach.

b. Design of the lee-side cover layer is based on the extent of wave overtopping, waves and surges acting directly on the lee slope, porosity of the structure, and differential hydrostatic head resulting in uplift forces which tend to dislodge the back slope armor units. If the crest elevation is established to prevent possible overtopping, the weight of armor units on the back slope cover layer should be less to reflect the lesser wave action on the lee side and porosity of the structure. When overtopping is anticipated, primary armor units should generally extend down the back slope to -0.5H below the minimum still water level. When both side slopes receive similar wave action (as with groins or jetties), both sides should be of similar design.

4-13. Example of Preliminary-Design Details.

a. General. The selected structure is a rubble-mound jetty trunk, with quarrystone armor, first and second stone underlayers, stone core, and stone bedding layer. The structure will be subjected to similar wave action from both sides and is to be designed for no overtopping.

b. Wave Characteristics. The design wave is 17 feet high with a period of 12 seconds. Waves are of the breaking type with an angle of incidence equal to 90 degrees.

c. Water Depths and Still-Water Levels. The depth of water at the toe, measured from National Geodetic Vertical Datum (NGVD), is 15 feet. The design still-water level is +6 feet.

d. Quarry Capability. The largest rough angular stones that can be obtained from the selected quarry in sufficient quantities have an average weight of 25 tons. The specific weight is 167 pounds per cubic foot.

e. Determination of Optimum Armor-Unit Weight and Slope Combination. Since the quarry for this project can provide only 25-ton armor units, the steepest slope for which 25-ton armor units will be stable under attack of 17-foot breaking waves will be the optimum solution. Equation 4-1 can be rearranged as

$$\cot \alpha = \frac{\gamma_a H^3}{W_a K_D (S_a - 1)^3}$$

The structure is assumed to be located on the seacoast, therefore γ_w = 64 pounds per cubic foot. The stability coefficient, obtained from Table 4-2 is equal to 2 . Substituting into the above equation and solving for cot α we obtain

$$\cot \alpha = \frac{(167)\ (17)^3}{(50{,}000)\ (2)\ (167/64 - 1)^3} = 1.97$$

Therefore, slopes of 1V:2H will be used.

f. Runup and Selection of Crest Elevation. The optimum crest elevation that will satisfy the no overtopping criterion is the lowest elevation that will prevent all but minor overtopping. This elevation is equal to the sum of the design still-water level (+6 feet NGVD) and the vertical height of the wave runup, caused by the selected design wave. The runup, obtained from the SPM (item 132), is 20 feet. Thus, a crest elevation of +26 feet NGVD is selected.

g. Armor-Unit Weight for the Crest. The weight of armor units on the crest is the same as on the side slopes, i.e., W_a = 25 tons with a specific weight of 167 pounds per cubic foot.

h. Crest Width. When the structure is designed for no overtopping, minimum crest width should equal the thickness of three layers of the armor on the seaside slope. Minimum crest width is obtained from equation 4-2.

$$B = 3k_\Delta \left[\frac{W_a}{\gamma_a}\right]^{1/3}$$

The value of k_Δ is found in table 4-3 and is equal to 1.0. Substituting this value, W_a = 50,000 pounds and γ_a = 167 pounds per cubic foot we obtain

$$B = 3\ (1)\ \left[\frac{50{,}000}{167}\right]^{1/3} = 20.07 \text{ feet}$$

Therefore, a crest width of 20 feet will be used.

i. First Underlayer Stone Weight. Stability of the armor units is not affected appreciably by the size of stones in the first underlayer unless the

material is so small that it will be pulled into voids of the cover layers. This can be prevented to a sufficient extent by use of graded first underlayer stone with a $(W_a)_{50}$ weight equal to 1/10 the weight of the armor stone. Thus weight of the first underlayer stones is 25 tons/10 = 2.5 tons.

j. Second Underlayer Stone Weight. Weight of the second underlayer stone can be as small as 1/20 the weight of the first underlayer. Thus, in this case, the $(W_a)_{50}$ weight of the graded second underlayer stones is 2.5/20 = 0.125 ton = 250 pounds.

k. Core Material Weight. The $(W_a)_{50}$ weight of the graded core material should be about 1/10 that of the second underlayer stone, i.e., about 25 pounds.

l. Thickness of Armor Unit Layer and Underlayers. Layer thicknesses can be determined from equation 4-3 using $n = 2$, $k_\Delta = 1.0$, and $\gamma_a = 167$ pounds per cubic foot.

Armor Unit Layer:

$$r = (2)\ (1)\ \left[\frac{50,000}{167}\right]^{1/3} = 13.4 \text{ feet}$$

First Underlayer:

$$r = (2)\ (1)\ \left[\frac{5,000}{167}\right]^{1/3} = 6.2 \text{ feet}$$

Second Underlayer:

$$r = (2)\ (1)\ \left[\frac{250}{167}\right]^{1/3} = 2.3 \text{ feet}$$

m. Bedding Layer. For this example the bottom material is sand, therefore, a bedding layer designed as a filter layer should be used to prevent waves and currents from removing sand through voids in the structure. The filter should be 2 feet thick and meet requirements discussed in paragraph 4-10g.

4-14. Sealing Rubble-Mound Jetties or Breakwaters.

a. Jetties are usually constructed impermeable and to the necessary elevation for retaining longshore drift. Impermeability can be obtained by such means as using a high core composed of fine materials, driving steel piling before the core is placed, or forming a diaphram with geotechnical fabrics. There are cases where it is necessary to make existing structures impermeable. This can be accomplished by sealing with asphaltic or concrete grout or using explosives to pulverize the inner core, thereby decreasing structure permeability.

b. Jetties at the entrance to Mission Bay, California, contained a sandtight core extending from the bottom to mean lower low water (mllw). The structure was sealed by drilling 2-1/2-inch holes on 6-foot centers along the center line of the jetty to the top of the core section. Grout was pumped through 1-1/2-inch nozzles to form a cone extending from the core stone to an elevation of +6 feet mean lower low water (mllw). These cones overlapped to form a sandtight seal. Test borings should always be made to ensure that sealing is complete. The grout mixture, per cubic yard, was comprised as follows:

Constituent	Weight, pounds
Sand	2,000
Cement	752
Illite clay	400
Water	537
Calcium chloride	16

Further details are contained in item 134. One disadvantage of sealing is increased back pressures.

c. The south jetty at Galveston Harbor, Texas (item 133), is an example of a structure sealed with asphaltic concrete. Core and capstones were consolidated to 1.5 feet below mean Gulf level (mGl) by hot asphaltic concrete forced into interstices of the rubble by steam-driven and heated vibrators and tampers especially designed for the work. The section of this jetty where asphaltic concrete was used is protected by sand accretion and is not subject to severe wave action.

d. The outer end of the south jetty at the mouth of the Columbia River, where waves 40 feet high have been reported, was impregnated in 1936 with 12,737 tons of a hot mixture of asphalt mastic (85 percent sand and 15 percent

asphalt) in an attempt to stabilize the structure and prevent end-raveling of the jetty stone by storm waves running across the head. While computations and later observations indicate that the asphaltic mix completely filled the voids to about low-water level (26 feet below the crest), the measure did not prevent breakdown of the outer end of the jetty above low-water level. The raveling continued at a rapid rate. When the 400 degree Farenheit asphaltic mix was placed in water, it generated steam, then bubbled, and finally disintegrated; it was found that the hot mix could not be placed successfully below the waterline.

e. The use of sealers to fill voids in rubble-stone structures has not proven to be effective in consolidating the core and capstone to give increased stability to the structure against wave action.

4-15. Quality Control Specification Requirements for Construction Materials.

a. General. Specifications should include the following information:

(1) Descriptions of physical properties including chemical and biological stabilities in the marine environment.

(2) Testing procedures in conformance with standards recommended by such groups as the American Society of Testing Materials (ASTM), US Bureau of Reclamation (USBR), American Concrete Institute (ACI), and the Corps of Engineers (CE).

(3) Ranges and gradings of size and mass for materials of heterogeneous nature such as sand and gravel or quarrystone.

(4) Quality control standards, procedures of implementation, and expected results.

(5) Descriptions of construction programs, including inspection standards, practices, and testing frequency.

b. Foundation Fill, Filter Layer, Core, and Scouring Blanket.

(1) Stone should be within the size range specified and the material should be well blended.

(2) Stones with the largest dimension, greater than three times the least dimension, should not constitute more than 10 percent of the total.

(3) Materials should be inert to chemical and biological degradations in sea water.

(4) The following standard tests are suggested to establish material durability:

(a) Abrasion test: ASTM C-535 or equivalent.

(b) Toughness test: ASTM C-170 or equivalent.

(c) Hardness test: ASTM C-235 or equivalent.

(d) Apparent specific gravity and absorption test: ASTM C-127 or equivalent.

c. Underlayer Stone. Underlayer materials should meet the following requirements in addition to those described in item b:

(1) Stones with their largest dimension greater than three times the least dimension should be rejected.

(2) The material should have adequate freezing and thawing resistance for the range of anticipated weather conditions.

d. Armor Stone. Stability as a whole depends primarily on the armor's ability to withstand dynamic loadings induced by the hostile ocean environment. Armor stone should meet, in addition to the requirements described in items b and c, the following conditions:

(1) The stones should have high specific gravity and low absorption.

(2) Materials should be able to withstand design impact conditions.

e. Concrete Armor Units. Concrete material in armor units should follow design and construction guidance in EM 1110-2-2000.

(1) The specified 28-day compressive strength of concrete shall be 5,000 pounds per square inch.

(2) Materials shall conform to current ASTM, ACI, and Corps specifications and codes.

(3) Individual units should be able to withstand design impact loadings.

4-16. Rehabilitation. Structures which have deteriorated to the extent that the cost of repair is beyond normal operation and maintenance funding capabilities should be rehabilitated. Design studies should generally follow the procedures set forth for new structures. The cause of damage and the structure's damage history should be available to enhance the selection of design conditions.

4-17. Maintenance.

a. The extent of maintenance required by rubble-mound structures will

depend primarily on the severity of wave action to which they are exposed. Operational plans should include annual inspections and special inspections after all major storms. Repairs should be initiated if damage is extensive enough to impair a structure's efficiency or reduce its ability to resist future storms.

b. The concept of designing a rubble-mound breakwater for zero damage is unrealistic, because a definite risk always exists for the stability criteria to be exceeded in the life of the structure. Table 4-4 shows results of damage tests where $H/H_{D=0}$ is a function of the percent damage, D, for various armor units. H is the wave height corresponding to damage D . $H_{D=0}$ is the design wave height corresponding to 0 to 5 percent damage, generally referred to as the no-damage condition.

c. Information presented in table 4-4 may be used to estimate anticipated annual repair costs, given appropriate long-term wave statistics for the site. For illustrative purposes, assume we have designed a breakwater that requires 1,000 tons of rough quarrystone per 100 lineal feet of structure and the average annual frequency of exceedance of H for $H/H_{D=0}$ values of 1.08, 1.19, 1.27, and 1.37 are 0.5, 0.4, 0.2, and 0.05, respectively. Probabilities of $H/H_{D=0}$ values in excess of 1.37 are assumed to be insignificant. Referring to table 4-4 and using the mean damages of the ranges presented for the various values of $H/H_{D=0}$, and the incremental average annual frequencies of exceedance as summarized in table 4-5, the expected weight of stone that will need replacement (per 100 feet of breakwater) is

$$w = 1,000\ (0.0769) = 76.9 \text{ tons}$$

Assuming it costs \$100 per ton to replace the stone, the expected annual maintenance cost is (76.9 tons) (\$100/ton) = \$7,690 per 100 lineal feet of structure per year.

Table 4-4. $H/H_{D=0}$ as a Function of Cover-Layer Damage and Type of Armor Unit[a].

Unit		Damage (D), Percent 0 to 5	5 to 10	10 to 15	15 to 20	20 to 30	30 to 40	40 to 50
Quarrystone (smooth)	$H/H_{D=0}$	1.00	1.08	1.14	1.20	1.29	1.41	1.54
Quarrystone (rough)	$H/H_{D=0}$	1.00	1.08	1.19	1.27	1.37	1.47	*1.56*[b]
Tetrapods and Quadripods	$H/H_{D=0}$	1.00	1.09	1.17[c]	1.24[c]	1.32[c]	1.41[c]	1.50[c]
Tribar	$H/H_{D=0}$	1.00	1.11	1.25[c]	1.36[c]	1.50[c]	1.59[c]	*1.64*[c]
Dolos	$H/H_{D=0}$	1.00	1.10	*1.14*[c]	*1.17*[c]	1.20[3]	*1.24*[c]	*1.27*[c]

(a) Breakwater trunk, n = 2, random-placed armor units, nonbreaking waves, and minor overtopping conditions.
(b) Values in *italics* are interpolated or extrapolated.
(c) CAUTION: Tests did not include possible effects of unit breakage. Waves exceeding the design wave height conditions by more than 10 percent may result in considerably more damage than the values tabulated.

Table 4-5. Determination of Percentage of Armor Stone Expected to be Replaced Annually

$H/H_{D=0}$	D	Avg D	AAFE	ΔAAFE	AVG D X ΔAAFE	Σ
1.37	0.25	—	0.05	—	—	—
		0.2125		0.15	0.0319	0.0319
1.27	0.175		0.20			
		0.150		0.20	0.0300	0.0619
1.19	0.125		0.40			
		0.100		0.10	0.0100	0.0719
1.08	0.075		0.50			
		0.050		0.10	0.0050	0.0769
1.00	0.025	—	0.60	—	—	—

NOTE: Percentage to be replaced is equal to the summation of the products of the average damages and incremental average annual frequency of exceedances, (AAFE).

CHAPTER 5

DESIGN OF VERTICAL WALL STRUCTURES

5-1. Sheet-Pile Structures. A sheet-pile structure consists of a line of piles engaged or interlocked to form a continuous wall. Piling is usually of steel, reinforced concrete, timber, or other materials. Choice of material will depend on relative cost, suitability for the intended use, and ability to resist lateral pressures. The cost of withdrawal and salvage value should be considered in the case of temporary works. For further design guidance, EM 1110-2-2906 should be consulted.

5-2. Steel Sheet Piles. Steel sheet piles are used for breakwater construction in three basic ways: (a) a single line of piling; (b) two parallel rows of piling connected by crosswalls or tie rods, and with sand or gravel fill between the walls; and (c) cellular units having either circular or semicircular sidewalls and crosswalls filled with sand or gravel. The last two types of construction are usually capped with large stones, a concrete slab, or bituminous paving. Corrosion protection should be provided on all steel sheet-pile structures.

a. Single-Wall Sheet Piles. The single-wall type is either buttressed on the harbor side by short lines of piles driven perpendicular to the main line, as shown in figure 5-1, or the piling is reversed to give a deep section. On the straight-wall type, wales are placed near the pile tops. They may be channel irons or heavy timbers bolted to each pile. Since stability of the single-wall type of structure is dependent upon its strength as a cantilever beam, deep web sections should be used. The penetration necessary to develop the required amount of resistance to cantilever action is governed by the wave forces present and the type of bottom materials. The necessary depth of penetration varies considerably with type of material; thus, a careful study should be made of the bed material.

b. Double-Wall Sheet Piles.

(1) Where steel sheet piling is used in depths that impose forces beyond its strength to resist as a cantilever, an adequate system of bracing must be provided. This is usually accomplished by constructing two walls approximately as far apart as the depth of the water. Each wall is stiffened with wales and attached to the other wall with tie rods. Further support can be provided by crosswalls of the same material at appropriate distances, which divide the breakwater into a series of bottomless cells or boxes. For further stability, the boxes are filled with stone or sand and capped with concrete, asphalt, or large stones. A reinforced concrete or asphalt cap is preferable as a covering for sand since it prevents loss of material by wave overtopping. Inspection manholes should be provided in the cap at regular intervals so that additional fill material can be added whenever needed.

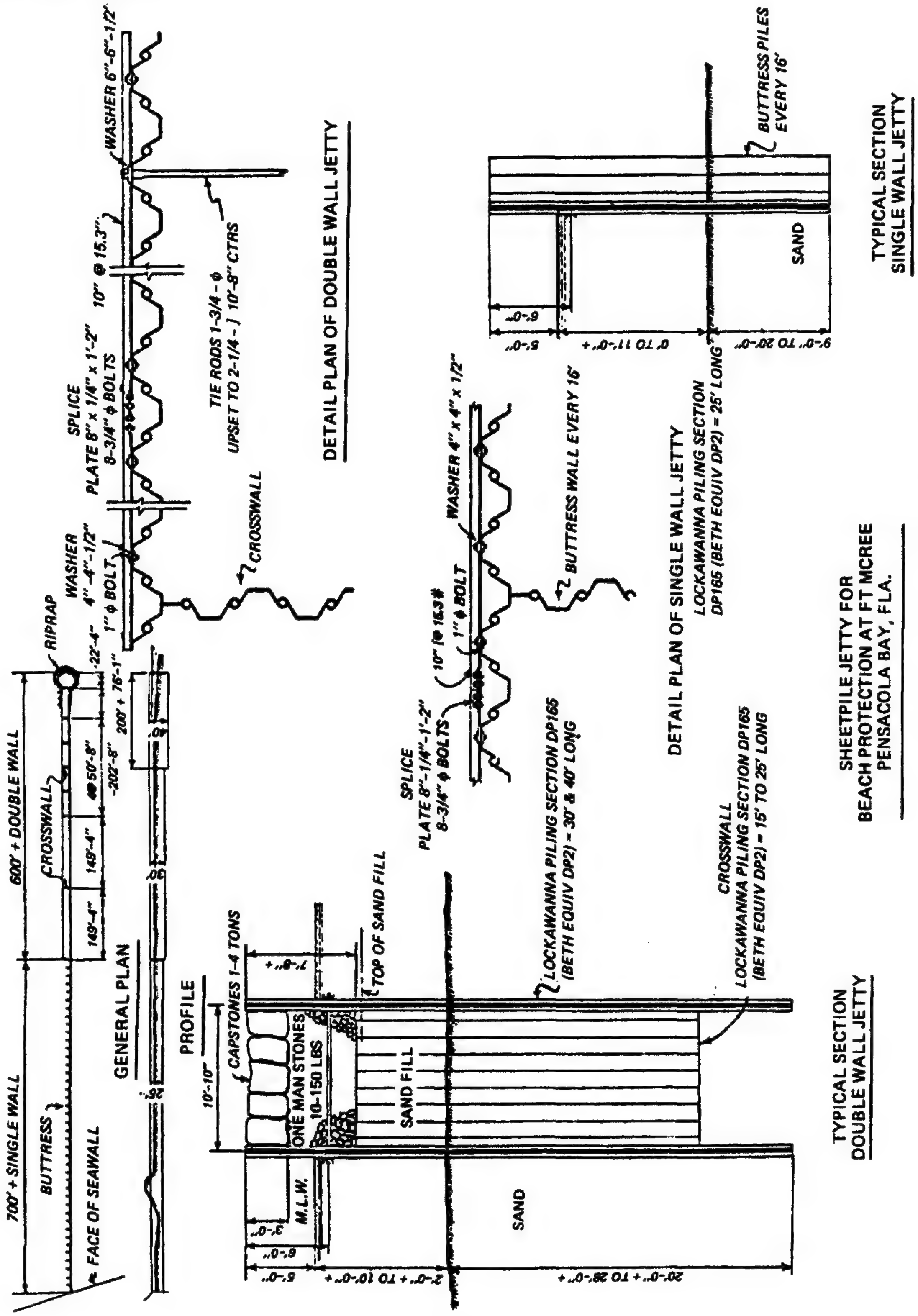


Figure 5-1. Typical steel sheet-pile jetty

(2) Experience has shown that the inside wales are preferable to outside wales; placing the wales inside protects them from the wave action and impact loadings from floating ice or other debris. Wales or other fixtures that tend to hold moisture and corrode should be located above high tide or below low tide.

c. Cellular Sheet Piles.

(1) When the breakwater is to be constructed in deep water, the use of underwater tie rods and wales becomes important: any system which requires the extensive use of divers is likely to be prohibitive in cost. To avoid this problem and to provide greater stability, cellular-type structures can be considered.

(2) Two types of cellular structures are currently used. The diaphragm-type, illustrated in figure 5-2, consists of a series of arcs connected to cross-diaphragm walls by means of fabricated Y-pieces. The legs of the Y-pieces form three 120-degree angles, making the tension in the cross-walls and arcs equal. The average width of the diaphragm type shown is 0.9 of the outside width.

(3) The circular type of breakwater, shown in figure 5-3, consists of a series of complete circles connected by shorter arcs, which are joined to the circles by means of fabricated T-pieces. As the T-pieces are usually manufactured at a 90-degree angle, it is imperative that the two sets of circles be orthogonal; the distances and radii indicated in figure 5-3 give right-angle intersections of the circles. The average width of the circular type shown is 1.7 times the radius of the circular arc.

5-3. Timber Sheet Pile.

a. Timber sheet piling is used for breakwater or jetty construction in areas subject to only moderate wave action and in relatively shallow depths. For saltwater use, timber must be pressure-treated as protection against marine borers. Physical properties of the various kinds of woods suitable for structural purposes are described in timber engineering textbooks. The design of timber sheet cantilever walls follows the same procedures as for other materials.

b. The most common type of timber sheet piling is known as Wakefield piling, shown as Type C in figure 5-4. This type, which is usually made on the job, consists of three thicknesses of plank with the middle plank offset to form a tongue and groove. The tongue-and-groove shape is sometimes made from a single timber. However, considering the size of timber necessary, waste involved, and added expense of milling the tongue-and-groove, this type is considerably more expensive than the Wakefield pile. In addition, tongue-and-groove piling is more susceptible to twisting and warping. Where a watertight fit between piles is of secondary importance, a plain rectangular pile is quite often used.

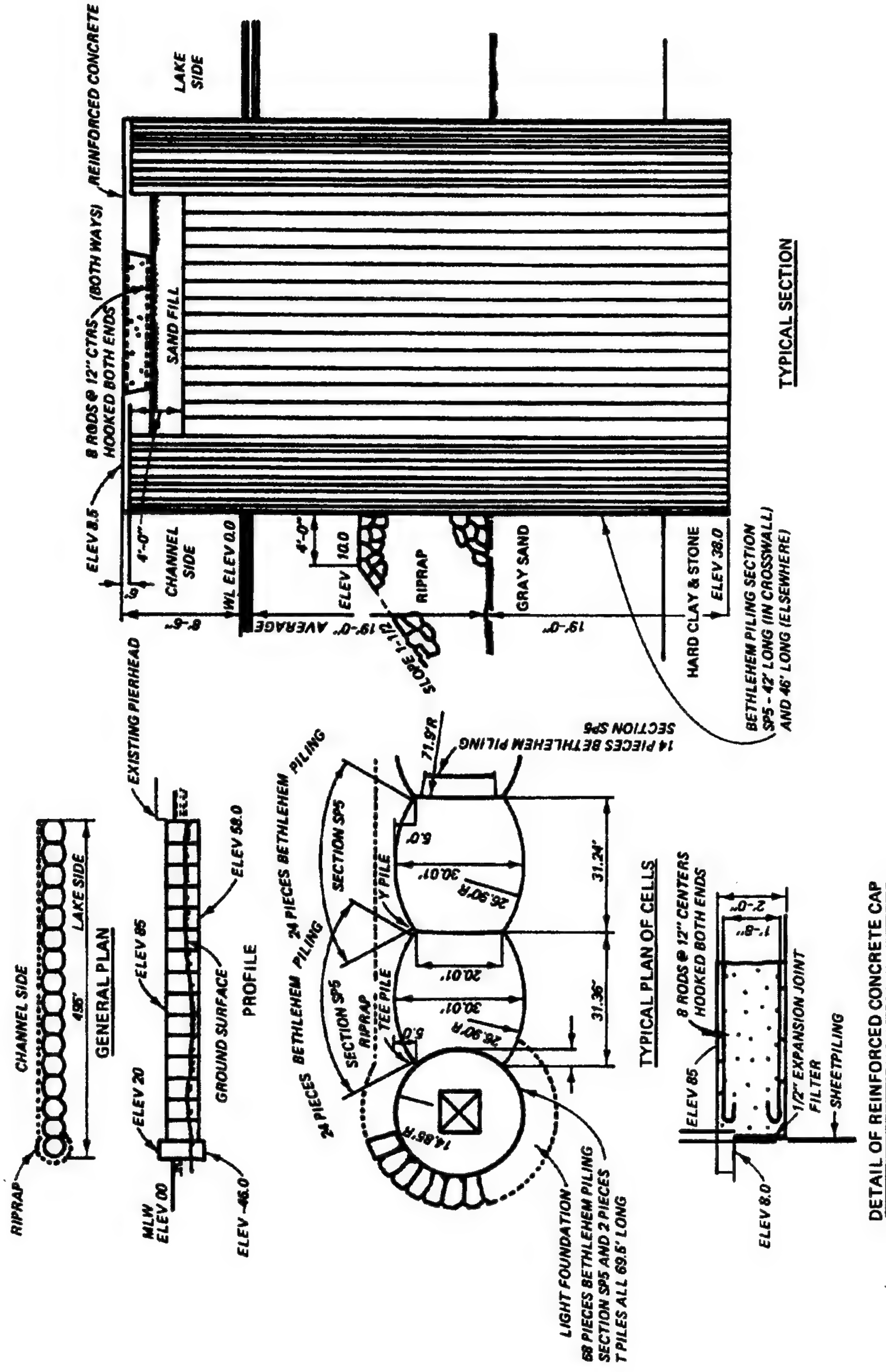


Figure 5-2. Diaphram type of cellular breakwater

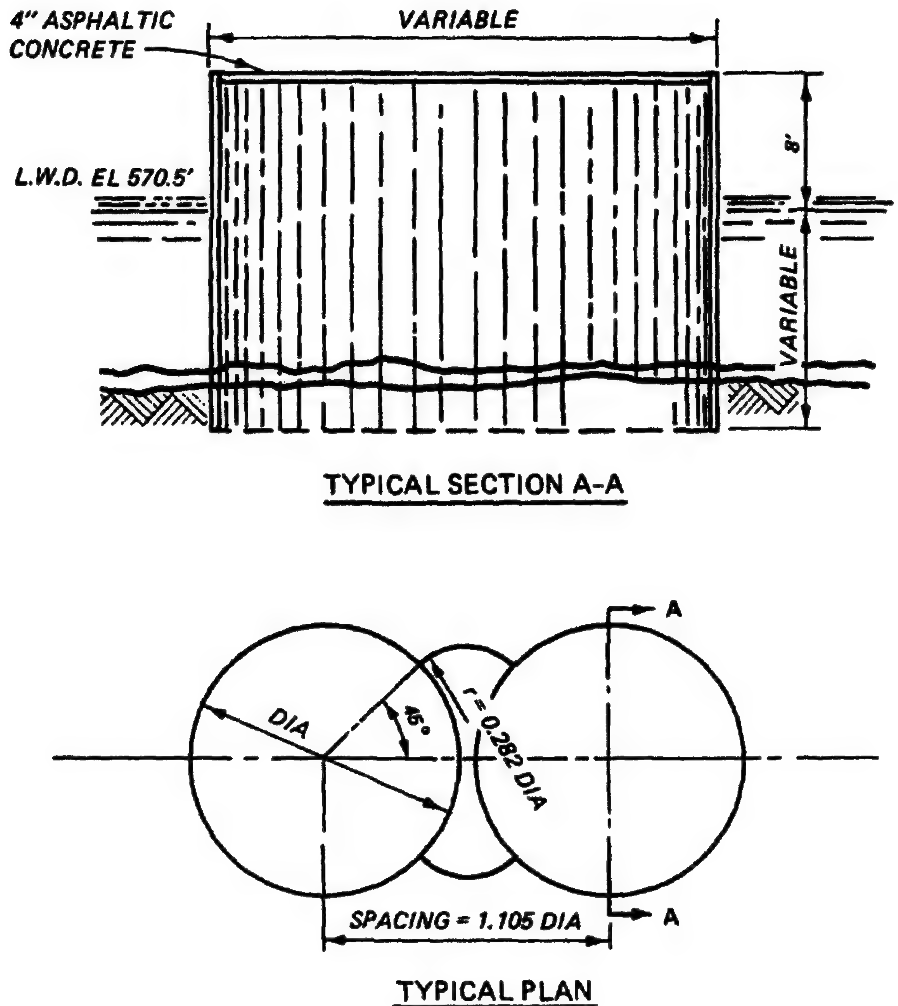


Figure 5-3. Circular type of cellular breakwater

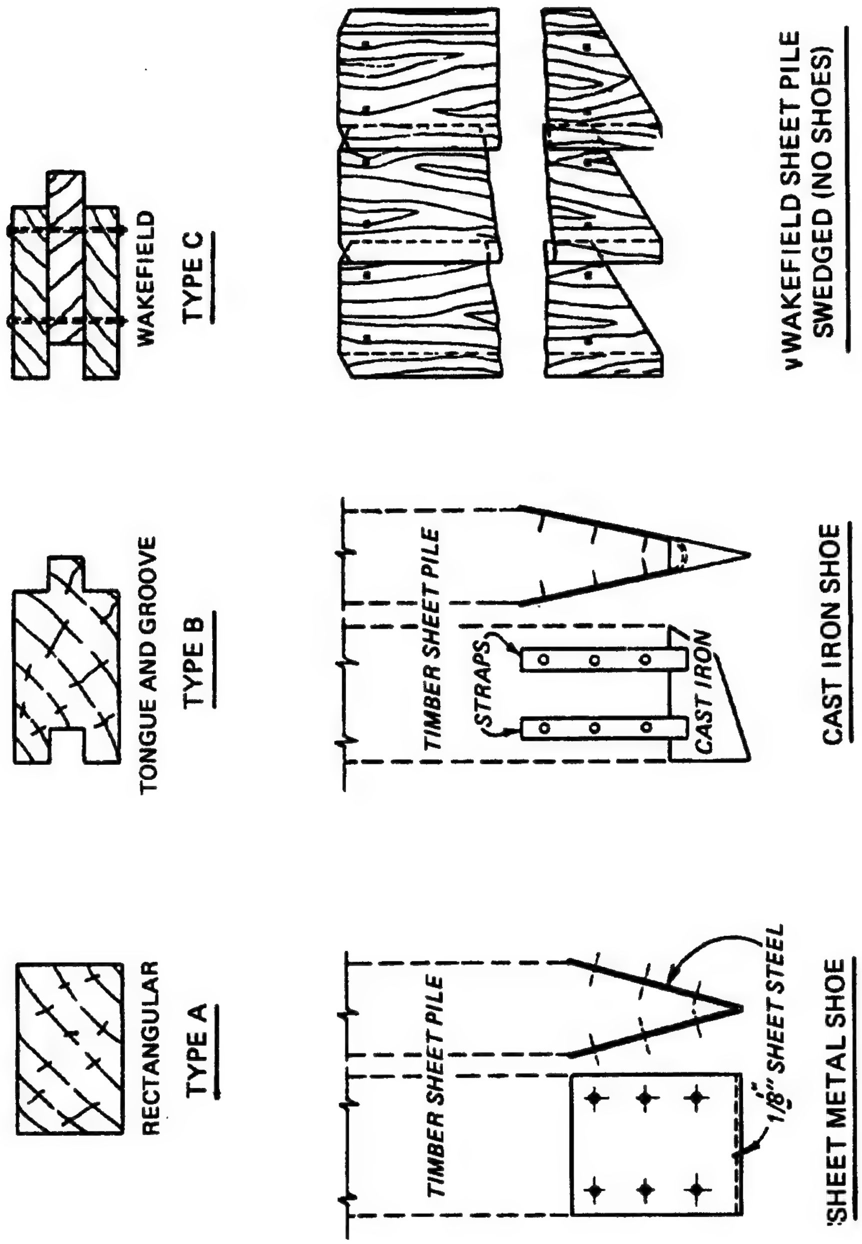


Figure 5-4. Typical timber sheet pile sections

5-4. Reinforced Concrete Piling.

a. If the forces which must be resisted have already been determined, pile dimensions, sizes, and spacing of the reinforcing bars are determined through application of ordinary reinforced concrete design principles. Depending upon the driving conditions, sheet metal or cast iron shoes can be fitted during the pouring operations. In order to ensure against corrosion, care should be taken in detailing the rebar so that an imbedment depth of at least 2 inches is obtained.

b. Typical sections of reinforced concrete piles are shown in figure 5-5. Special consideration should be given to the concrete composition when the structure is placed in saltwater, water contaminated by strong industrial residues, or in regions subjected to severe ice conditions.

c. Depending upon the type of structure desired, concrete pile forms can be constructed to obtain almost any type of shape of compression interlocking. Tension interlocks consisting of cast-in-place metal strips should be avoided because of concrete's low tensile strength. However, piles of this nature have been used as crosswalls between parallel rows of piles. The fill material between the outer rows causes the crosswalls to be in tension. Where the individual piles are securely held in position either by wales or a cast-in-place top covering, a satisfactory degree of water-tightness can be obtained by grouting between specially designed interlocks.

d. Concrete sheet piling should be specified using Guide Specification CEGS 02366, Precast Concrete Piling, or CEGS 02362, Prestressed Concrete Piling, as applicable. The concrete should be resistant to abrasion and not subject to disintegration when exposed to air, seawater, or freezing and thawing.

5-5. Wave Force Computations.

a. Wave forces exerted on vertical wall structures can be distinguished as being due to either nonbreaking, breaking, or broken waves. Whether a structure is subject to a single wave type or a combination of wave types depends on the wave climate, water depth, foreshore slope, and structure geometry.

b. The force due to nonbreaking waves is essentially hydrostatic. Sainflou's method or the modified Sainflou method, also referred to as the Miche-Rundgren method (item 132), is generally considered adequate for the vertical wall case. Figure 5-6 shows the wave pressure distribution according to the Sainflou method. ABED is the pressure diagram of the surface pressure due to wave action, DEC is the still-water pressure diagram, P is the value of the pressure due to wave action at the seabed, and h_o is the rise of the mean level of the clapotis (standing wave) formed due to the reflecting wave. Sainflou's equation for peak pressure involves hyperbolic trigonometric functions. The Miche-Rundgren method approximates the pressure distribution by a straight line as shown in figure 5-6. In this case, the only quantities

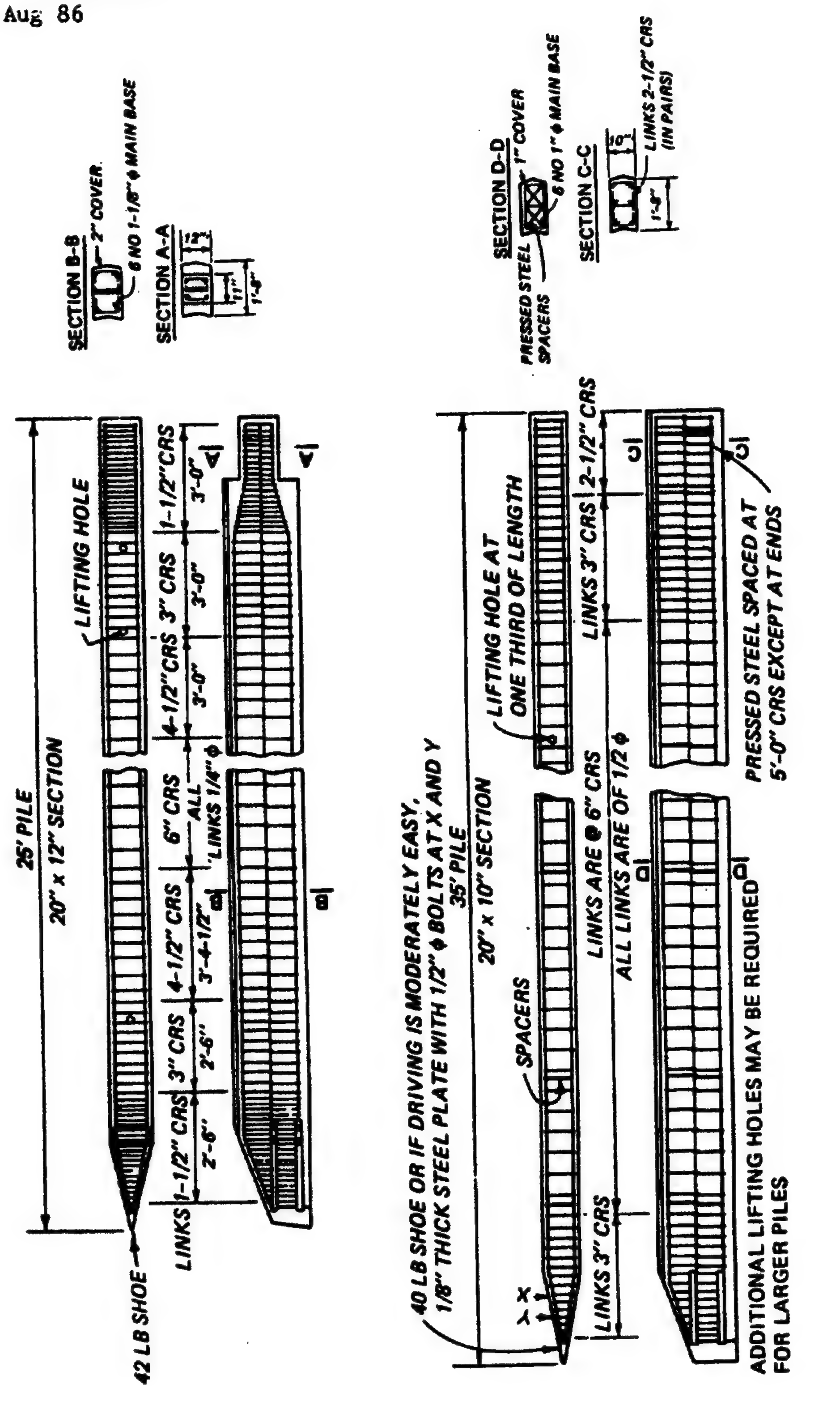


Figure 5.5 Typical reinforced concrete sheet pile sections

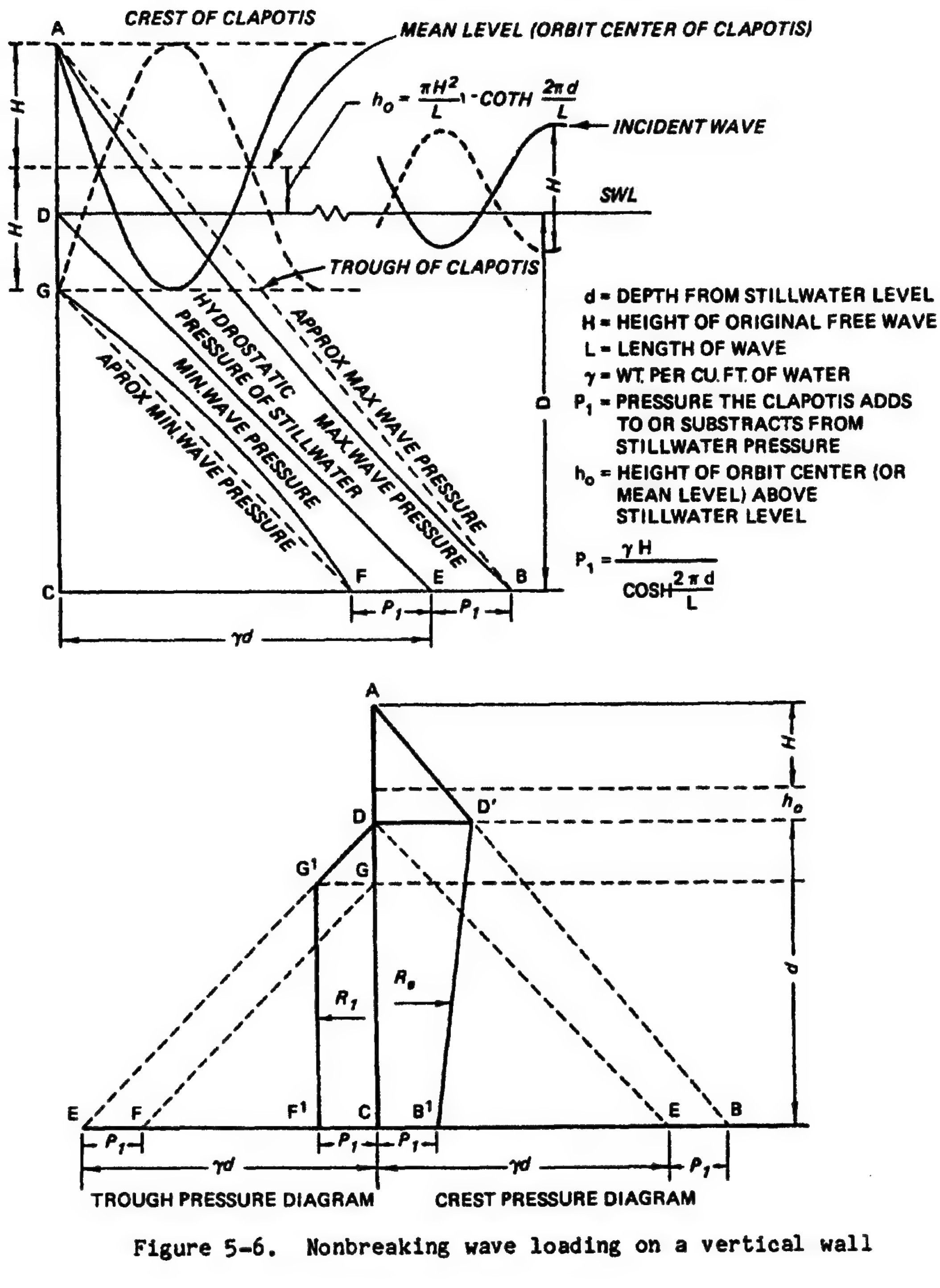


Figure 5-6. Nonbreaking wave loading on a vertical wall

which must be evaluated before the diagram can be drawn are the values of P_1 and h_o. These values are:

$$P_1 = \frac{1 + x}{2} \left[\frac{\gamma H_i}{\cosh(2\pi d/L)} \right] \qquad (5\text{-}1)$$

$$h_o = \frac{\pi H^2}{L} \coth \frac{2\pi d}{L} \qquad (5\text{-}2)$$

where

x = wave reflection coefficient (1.0 for vertical wall)

γ = specific weight of seawater

H = wave height

L = wave length

d = water depth

The corresponding resultant forces (R) and moments about the base (M) are given, respectively, for the maximum crest level (subscript e) and the maximum trough level (subscript i) by the following equations:

$$R_e = \frac{(d + H + h_o)\ \gamma d + P_1)}{2} - \frac{\gamma d^2}{2} \qquad (5\text{-}3)$$

$$M_e = \frac{(d + h_o + H)^2\ (\gamma d + P_1)}{6} - \frac{\gamma d^3}{6} \qquad (5\text{-}4)$$

$$R_i = \frac{\gamma d^2}{2} - \frac{(d + h_o - H)\ (\gamma d - P_1)}{2} \qquad (5\text{-}5)$$

$$M_i = \frac{\gamma d^3}{2} - \frac{(d + h_o - H)^2\ (\gamma d - P_1)}{6} \qquad (5\text{-}6)$$

c. Waves breaking directly against the structure face sometimes exert high, short-duration, dynamic pressure that acts near the region where the crests hit the structure. At present, Minikin's equation (item 132) is widely used in the United States; in Japan, Hiroi's equation is generally accepted. The Minikin equation gives a pressure distribution (shown in figure 5-7a) with peak pressure at or near the still-water level; Hiroi's equation, on the other hand, assumes a uniform pressure distribution (shown in figure 5-7b). Minikin's equation yields considerably higher peak pressure than Hiroi's, although the resulting total forces given by these two equations are similar for shallow-water cases. Both equations overestimate the total force and overturning moment when the water depth gets deeper. Items 54 and 99 present alternative equations for computing wave loading. Based on these works, the following equations are recommended:

(1) Peak impact pressure (P_m).

$$P_m = 2.5\ \gamma\ H \text{ tons per square foot} \tag{5-7}$$

(2) Total force (F_t).

(a) If $H/L_o < 0.045$,

$$F_t = 3H + P_1 \text{ (Sainflou) tons per lineal foot} \tag{5-8}$$

(b) If $H/L_o > 0.045$,

$$F_t = 4H \text{ tons per lineal foot} \tag{5-9}$$

(3) Moment (M)

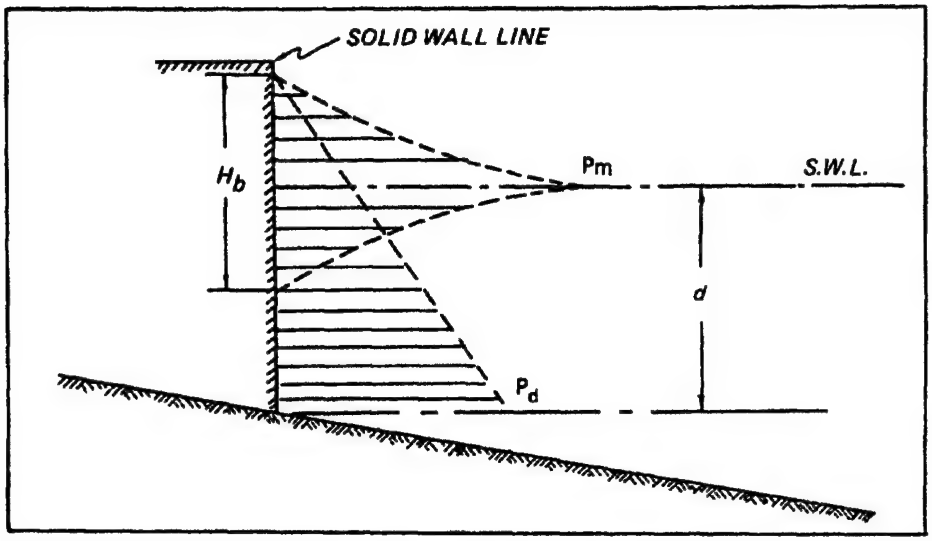
(a) If $H/L_o < 0.045$. (5-10)

$$M = 8H^2d \text{ ton-feet per lineal foot}$$

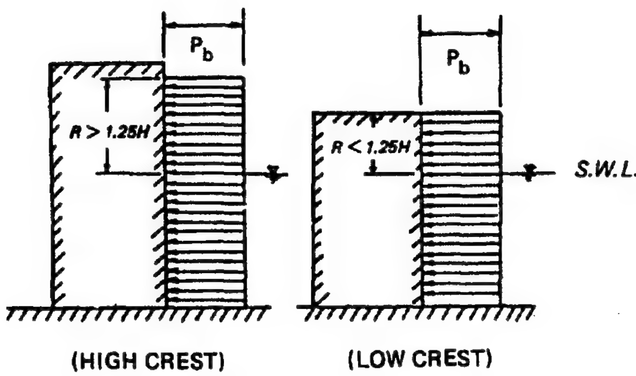
(b) If $H/L_o > 0.045$,

$$M = 12.5H^3 \text{ ton-feet per lineal foot} \tag{5-11}$$

5-6. Maintenance. Structures should be inspected on a periodic basis to identify maintenance needs. Timbers showing evidence of rot, decay, or marine borer intrusion can be replaced. Steel piling that is significantly weakened from corrosion may need to be replaced. Concrete structures should be inspected for cracking and sealed as needed to prevent intrusion of water.



a. Minikin Formula



b. Hiroi Formula

Figure 5-7. Breaking Wave Pressure Distribution on Vertical Walls

The overall stability of vertical wall structures is highly dependent on their toe stability; therefore, toe scour problems should be monitored and quickly corrected.

5-7. Rehabilitation. Structures that have sustained major damage from storms or have deteriorated to the extent that normal maintenance is impractical may require rehabilitation. If rehabilitation plans call for replacement of major structural features, the economic analysis should consider alternate types of structures, e.g., a timber structure might be most advantageously rehabilitated with steel sheet piling.

CHAPTER 6

DESIGN OF FLOATING STRUCTURES

6-1. Floating Breakwater Applicability.

a. Permanently fixed breakwaters generally provide a higher assurable degree of protection than floating breakwaters; however, they are expensive to construct. In deep water, a fixed breakwater may not be economically competitive with a floating breakwater, depending on the incident wave period. Floating breakwaters provide less protection, but they are less expensive and are movable from one location to another as required. A floating breakwater may be relatively easy to fabricate at a site where a rigid bottom-resting gravity structure would be completely infeasible.

b. Several major points exist in the consideration of a floating breakwater. The cost of a floating system is only slightly dependent on water depth and foundation conditions. Whereas the construction cost of a fixed rubble-mound breakwater increases exponentially with depth, a floating breakwater requires essentially the same structural features regardless of the water depth (except for mooring arrangements). The interference of a floating breakwater with biological exchange and with circulation and flushing currents essential for the maintenance of water quality is minimal (again depending on the incident wave period). The planform layout can be changed to accommodate changes in either seasonal or long-term growth patterns. Floating breakwaters appear to have greater multiple-use potential than fixed structures; i.e., they can be used as boat docks or boat mooring locations, and also serve as walkways.

c. Floating breakwaters, however, have some characteristics which must be weighed in their evaluation. The design of a floating breakwater system must be carefully matched to the site conditions, with due regard to the longer waves which may arrive from infrequent storms. The floating breakwater can fail to meet its design objectives by transmitting a larger wave than can be tolerated without necessarily suffering structural damage. Uncertainties in the magnitude and types of applied loading on the system, as well as lack of maintenance cost information, dictate conservative design practices which naturally increase the initial project cost. A major disadvantage is that floating breakwaters move in response to wave action and are thus more prone to structural fatigue.

6-2. Floating Breakwater Groups. At least 60 different floating breakwater configurations are recognized (items 78 and 117). Geometric and functional similarities among these various configurations allow for logical classification into basic groups based on fundamental features. These groups include the following breakwater types: pontoon, scrap tire, A-frame, tethered float, porous walled, flexible membrane, turbulence generator, and peak energy dispersion. Design of the pontoon and scrap tire types will be discussed herein. Items 56 and 142 describe the other groups in detail.

6-3. Operational Considerations. Certain fundamental operational aspects exist that are common to all types of floating breakwaters. These include the determination of the incipient wave conditions for performance considerations and the type of anchoring system to be developed for a particular location. The basic methods by which a floating breakwater reduces wave energy to produce a sheltered region include (a) reflection, (b) dissipation, (c) interference, and (d) conversion of the energy into mono-oscillatory motion. For effective reflection, the breakwater should remain relatively motionless and penetrate to a depth sufficient to prohibit appreciable wave energy from passing underneath. The structure could extend to the bottom and obstruct most of the water column, but it usually floats with a draft much less than the water depth. For short waves in the upper part of the water, deep draft is not needed; for long waves, deep draft may be desirable but again it is difficult to contend with the large mooring loads which may result. Optimization is often required between the wave attenuation aspects and mooring loading. Because of this turbulent dissipation of energy, forces in the mooring system are accordingly reduced.

a. Performance Evaluation. The generally accepted criterion for evaluating a breakwater's performance is the transmission coefficient C_t . This parameter is usually defined as the ratio of the transmitted wave height H_t to the incident wave height H_i , or

$$C_t = \frac{H_t}{H_i} \tag{6-1}$$

As with all breakwaters, the design of a floating breakwater is always site-specific. Waves favorably attenuated by a floating breakwater usually do not exceed 4 feet in height, and periods usually do not exceed 4 seconds; hence, for these relatively short-period waves, refraction and diffraction probably do not enter into the determination of the wave climate. If necessary, however, these effects can readily be incorporated into the design considerations. The wave length L is uniquely related to wave period for the water depth in which the wave is propagating as

$$L = \frac{gT^2}{2\pi} \tanh \frac{2\pi d}{L} \tag{6-2}$$

where

g = gravitational constant

T = wave period

d = local water depth

The peak waves, or the rare, extreme occurrences, are the parameters the structure must be designed to withstand. Once the incoming wave climate has been ascertained, the acceptable wave heights which can be tolerated in the sheltered area must be determined. When the acceptable transmitted wave has been determined, the design transmission coefficient is fixed.

b. Anchoring Systems.

(1) The type of system selected for anchoring a floating breakwater depends on the peak mooring forces estimated for the structure, the bottom conditions at the site, and the methods available for installing the anchor (item 51). The two most commonly used methods for anchoring any type of floating breakwater are the deadweight anchor and the pile anchor. Embedment anchors and screw anchors have had limited use, primarily because they have fairly short lengths and are difficult to install in firm marine soils.

(2) The deadweight anchor is usually a concrete block cast at the site. The design anchor weight W_t is determined by the forces available to cause movement and the degree of resistance produced by the static friction of the bottom conditions (mud, sand, or rock bottom). Based on a static analysis, the relationship between these variables is

$$W_t = \frac{F_t F_s}{\mu - \left(\frac{\mu \gamma_w}{\gamma_c}\right)} \tag{6-3}$$

where

F_t = lateral mooring line peakload

F_s = factor of safety

μ = coefficient of soil static friction

γ_w = unit weight of water

γ_c = unit weight of concrete in air

Deadweight anchors are usually positioned four to eight water depths from the structure.

(3) Anchor piles are designed by finding the ultimate lateral resistance of the pile-soil system and increasing the lateral mooring load F_t by a factor of safety F_s to determine the design lateral load on the pile. The ultimate lateral resistance of the anchor pile is reached when either the passive strength of the surrounding soil is exceeded or when the yielding moment of the pile section is reached. The short rigid pile case will

normally suffice for anchor piles for floating breakwaters. The short rigid pile is assumed not to bend when laterally loaded but will rotate about a point approximately one-third to one-quarter its length above the pile tip. Anchor piles are designed for the soil's ultimate lateral resistance rather than deflection of the pile head; hence, the design is predicated on sufficiently large deflection to develop the full passive resistance. This is defined as three times the Rankine passive earth pressure from the soil surface to the center of rotation. The expression for the ultimate lateral resistance of a short pile in a cohesionless soil is

$$F_t F_s = \frac{(\gamma_s D \ell^3 K_p)}{(2e + 2\ell)} \tag{6-4}$$

where

γ_s = unit weight of soil

D = pile diameter

ℓ = distance pile penetrates into the bottom

K_p = Rankine's coefficient of passive earth pressure $= \dfrac{(1 + \sin \phi)}{(1 - \sin \phi)}$

e = distance load is applied above the bottom

ϕ = internal friction of sand

(4) When the foundation soil conditions at the breakwater site are cohesive, the method presented in item 14 can be used to determine the ultimate lateral resistance of a rigid-pile anchor under lateral load. The distance the pile penetrates into the bottom is

$$\ell = 1.5D + f + q \tag{6-5}$$

where

$$f = \frac{(F_f F_s)}{(9c_u D)} \tag{6-6}$$

and

$$q = \frac{F_t F_s (e + 1.5D + 0.5f)^{0.5}}{(2.25D)} \qquad (6\text{-}7)$$

Here c_u is the undrained cohesive strength of the soil. The pile spacing as well as the deadweight anchor locations should be close enough to overcome the peak lateral forces exerted by the floating breakwater on the mooring lines.

6-4. Pontoon Floating Breakwaters.

a. General. To be effective as a breakwater, the motions of a floating structure must be of small amplitude so that the structure does not generate waves into the protected region. Although at resonance the generated waves can be out of phase with the transmitted waves (resulting in lower coefficients of transmission), the structure must respond to a spectrum of incident wave conditions. Hence, the design of a floating structure for resonance characteristics only is not satisfactory. Designers seek to achieve small wave transmission by incorporating (1) a large mass to resist the exciting forces and (2) a natural period of oscillation which is long with respect to the period of the waves (item 145). To obtain a long natural period, it is generally necessary to combine large mass with small internal elastic response of the entire system. A floating breakwater should also extend deep enough into the water so that little of the wave kinetic energy can be transmitted beneath the structure. To make the internal elasticity small and the mass large at the same time, the bulk of the breakwater should be below the water surface. A moored structure has an additional elastic restraining force due to the mooring lines, and the mass to be considered is the virtual mass which includes the added mass of the water. The simplest forms of floating breakwaters include pontoon structures, although various modifications to geometry have been investigated in an effort to optimize the mass (and ultimately the cost) of potentially viable alternative systems.

b. Single-Pontoon Floating Breakwaters.

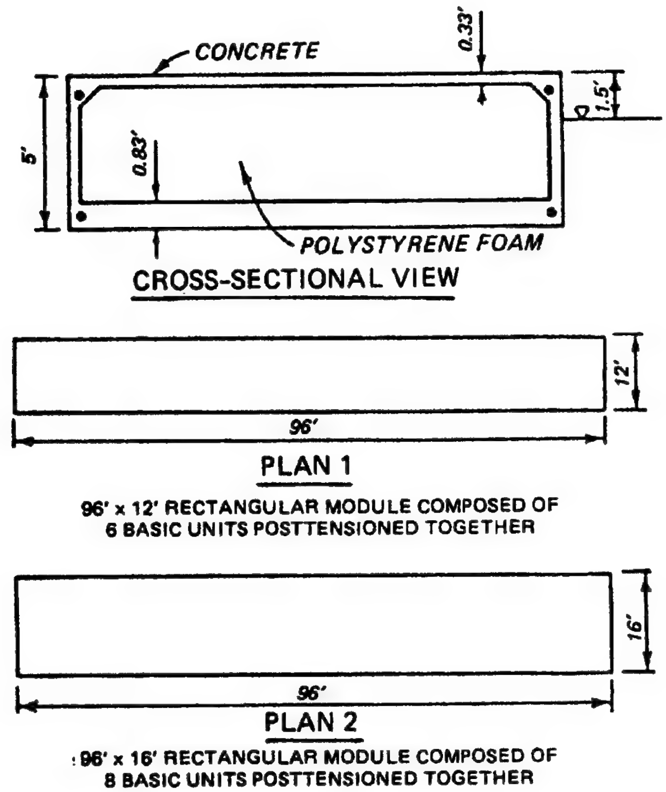
(1) The rectangular, prismatic (single) pontoon floating breakwater has been considered by several investigators either as a possible system or as a reference for comparison with other potential systems (items 9, 19, 25, 59, and 103).

(2) Three single-pontoon floats have been evaluated (item 25). Specific details of the various plans were as follows:

(a) Plan 1 was a 12- by 96-foot rectangular float with a draft of 3.5 feet. The prototype structure weighed 258,000 pounds and had a unit weight of 44.8 pounds per cubic foot. Plan 1 was modeled with a uniform cross-sectional structure, 1.2 feet wide by 9.6 feet long, weighing 252 pounds with a unit weight of 43.7 pounds per cubic foot.

(b) Plan 1A was identical to Plan 1 except that a 3.5-foot-high vertical barrier plate was added to the bottom of the structure's seaward face.

(c) Plan 2 was a 16- by 96-foot rectangular float with a draft of 3.5 feet. The prototype structure weighed 344,000 pounds and had a unit weight of 44.8 pounds per cubic foot. The Plan 2 model breakwater also had a uniform cross section, 1.6 feet wide by 9.6 feet long, weighed 335 pounds, and a unit weight of 43.7 pounds per cubic foot. The details of Plans 1 and 2 are shown in figure 6-1.



WEIGHTS AND UNIT WEIGHTS

CONFIGURATION	WEIGHT, LB	UNIT WEIGHT, LB/FT^3
PLAN 1	258,000	44.8
PLAN 2	344,000	44.8

Figure 6-1. Details of Plans 1 and 2 for a single-pontoon floating breakwater evaluated in two-dimensional (2-d) model tests for application at Olympia Harbor, Washington

(3) All the plans investigated utilized crossed anchor chains; i.e., beach-side anchor points on the breakwater were connected to seaside anchor points on the floor, and seaside anchor points on the breakwater were connected to beach-side anchor points on the floor. Wave attenuation tests were conducted in 25 feet of water with prototype wave periods of 2.5, 3.0, 3.5, 4.0, and 4.5 seconds. Test waves ranged in height from 1.5 to 3.5 feet and transmitted waves were measured one wavelength behind the structure. The two-dimensional transmission coefficients C_t for Plans 1, 1A, and 2 are presented in figure 6-2.

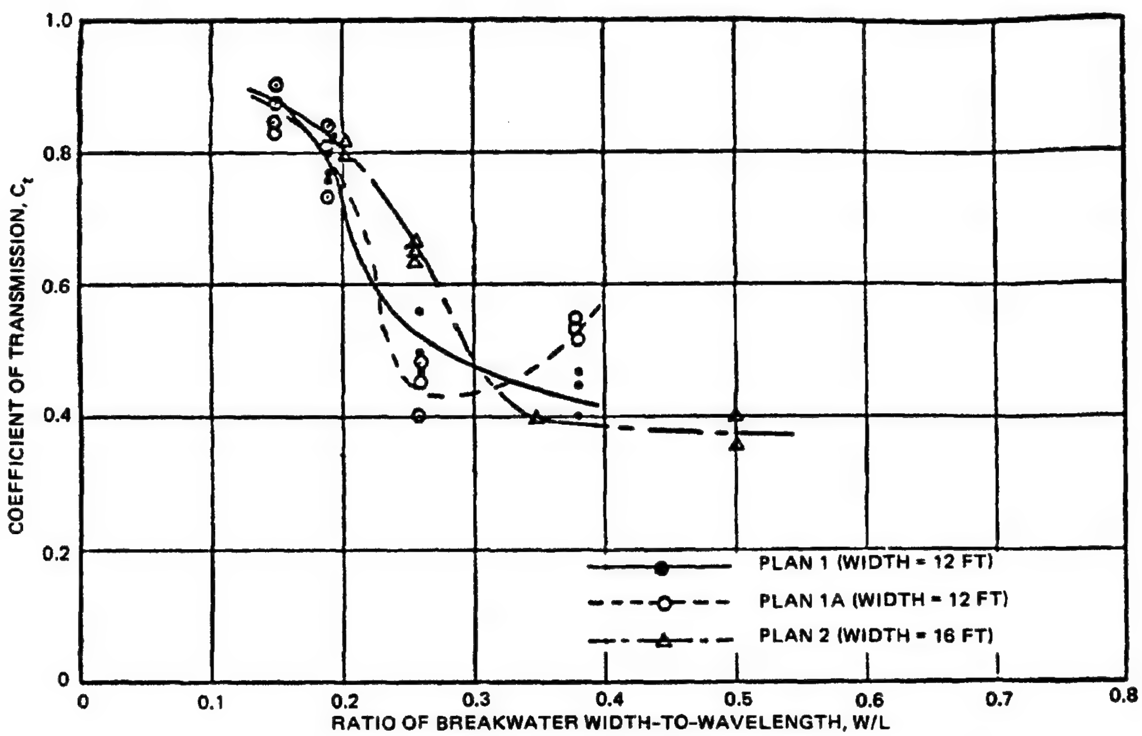


Figure 6-2. Effect of relative breakwater width on coefficient of transmission for a single-pontoon floating breakwater evaluated in two-dimensional model tests for application at Olympia Harbor, Washington

(4) Plans 1 and 1A test results afforded some interesting comparisons. Based solely on the physical dimensions of the structure, it is reasonable to assume that for the range of wave conditions tested, Plan 1A exhibited a slight increase in performance relative to Plan 1. Actually, Plan 1A exhibited slightly higher transmitted values for the 2.5-second wave period, slightly lower values for the 3.0-second wave period, and almost the same values for the 3.5-second wave period. The dynamic response of Plan 1A was significantly different from that of Plan 1. A decrease in roll and an

increase in heave was observed for all wave conditions, indicating that the mechanism of wave transmission was fundamentally different and accounting for the variations in transmitted wave heights. Based on these observations, it can be postulated that the decrease resulted because wave components generated by heave and sway motions were almost 180 degrees out of phase and tended to cancel each other. Since Plans 1 and 2 were both single-pontoon floats with widths of 12 and 16 feet, respectively, Plan 2 was expected to generally yield somewhat lower transmitted wave heights than Plan 1. Plan 2, indeed, exhibited a constant increase in performance relative to Plan 1 for W/L values greater than about 0.3; however, this improved performance was not discernible at smaller values of W/L .

c. Twin-Pontoon Floating Breakwater. Twin-pontoon floating breakwaters consist of rectangular cross sections which are rigidly connected at selected intervals. The open interior allows turbulent energy dissipation between the separate single-pontoon sections. The concept achieves wave attenuation primarily by reflection from a structure with a large radius of gyration which experiences only small displacements; turbulence plays a secondary role. A two-dimensional model was tested (item 45) to obtain wave attenuation characteristics and mooring forces for a catamaran-type (twin pontoon) breakwater proposed for Oak Harbor, Washington. (A schematic of the structure is shown in figure 6-3.) Tests were conducted of a 1:10-scale specifically to determine (a) the effectiveness of the proposed structure in reducing the existing wave heights, (b) the mooring forces for both the chain- and the pile-type mooring systems, (c) whether or not the proposed breakwater and mooring system would oscillate in resonance with the existing wave conditions, and (d) the natural period of oscillation of the proposed breakwater while unrestrained in still water. One module of the proposed breakwater was reproduced. In the model, the chain

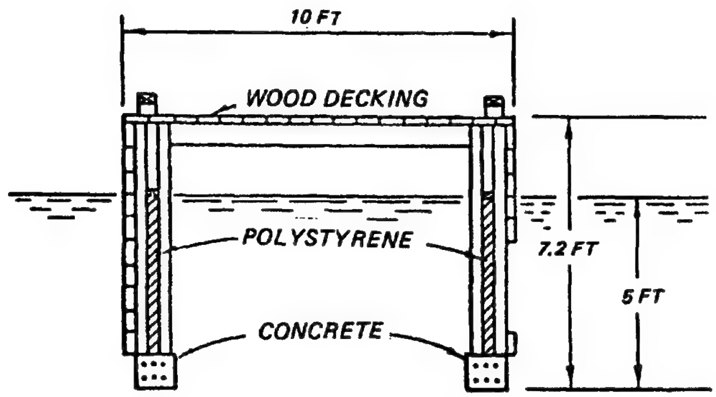


Figure 6-3. Two-dimensional model arrangement of catamaran-type floating breakwater, Oak Harbor, Washington

mooring system consisted of two anchor chains on each side of the breakwater module. Each chain was fastened to strain-gage measuring devices on the bottom of the test flume to measure the forces in the mooring lines. The pile mooring system consisted of a pile on each end of the module. These piles were strain-gaged to measure the seaside and harbor-side forces in the direction parallel to that of wave travel.

(1) Transmission tests with chain mooring system. Tests were conducted at 10- and 29.5-foot water depths for the selected wave conditions. The flotation depth of the modules at the 29.5-foot water depth was 5.0 feet; each of the four anchors had an initial tension force of about 2,200 pounds (approximately 100 pounds per foot of structure width perpendicular to the direction of wave travel). When the water level was lowered to the 10-foot depth, most of the anchor chains lay on the flume bottom; thus, the initial tension on the anchors was reduced to zero and the draft of the floating module was decreased to approximately 3.8 feet. Transmitted wave heights are presented in figure 6-4. These data indicate that the transmitted wave height varies more with wave period than with change in water depth. For an allowable transmitted wave height of 0.5 foot, proposed modules, using a chain mooring system, would not be adequate for incident waves greater than approximately 2.0 feet in height and approximately 2.5 seconds in period. During wave attack, the module oscillated about its longitudinal center line and at the same time rocked with the waves. Overtopping of the module began with lesser wave heights at the 29.5-foot depth because the initial tension in the chain restraints limited the upward motion of the module at this depth more than at the 10-foot depth. During the transmission tests, the module was not observed to be in resonance with any of the wave periods tested.

(2) Transmission tests with pile mooring system. Tests were performed at the 10- and 29.5-foot water depths to determine the effectiveness of the proposed catamaran-type breakwater with a pile mooring system. The flotation depth at both water depths was 5.0 feet. Results of the transmission tests with a pile mooring system are shown in figure 6-5. These data indicate that, with the exception of the 3.0-second wave period, the transmitted wave height again varied more with wave period than with change in water depth. The 3.0-second wave period at both depths caused the breakwater module to rock in such a fashion that larger transmitted wave heights were produced than had been anticipated, resulting from resonant action of the system. For a maximum incident wave height of 2.0 feet and an allowable transmitted wave height of 0.5 foot, a breakwater constructed of the catamaran-type modules would be inadequate for wave periods greater than 2.5 seconds.

(3) Anchor force tests with chain mooring system. For each chain, the peak anchor force was taken as the sum of the initial force placed in the anchor chain and the highest peak force that occurred for a given test condition. The average anchor force was taken as the sum of the initial anchor chain force and the average of the highest one-third of the peak anchor forces measured during a test. Anchor chain force data are shown in figure 6-6 as plots of the anchor force per foot of structure width versus

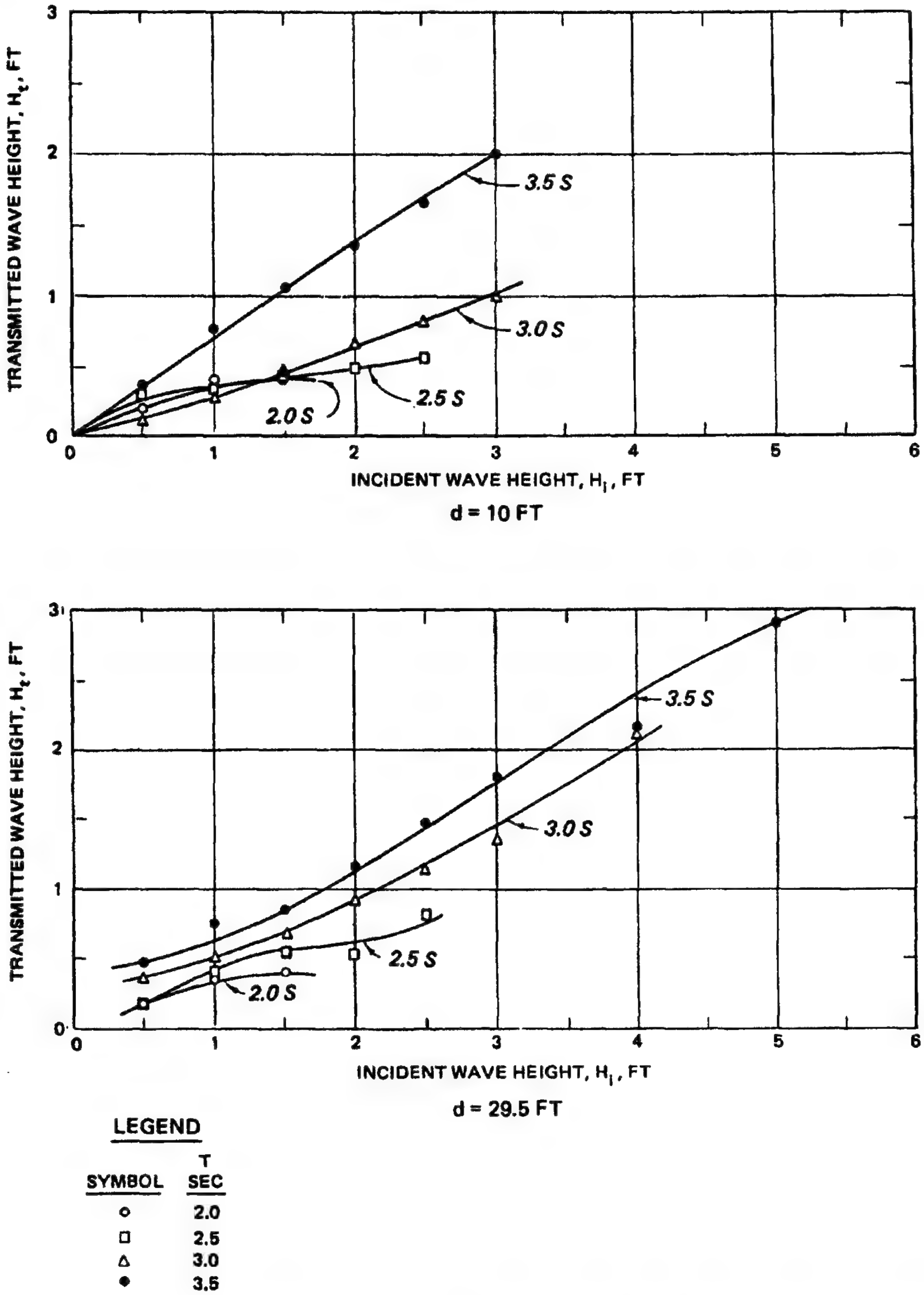


Figure 6-4. Wave transmission test results for the chain mooring system, catamaran-type floating breakwater, Oak Harbor, Washington

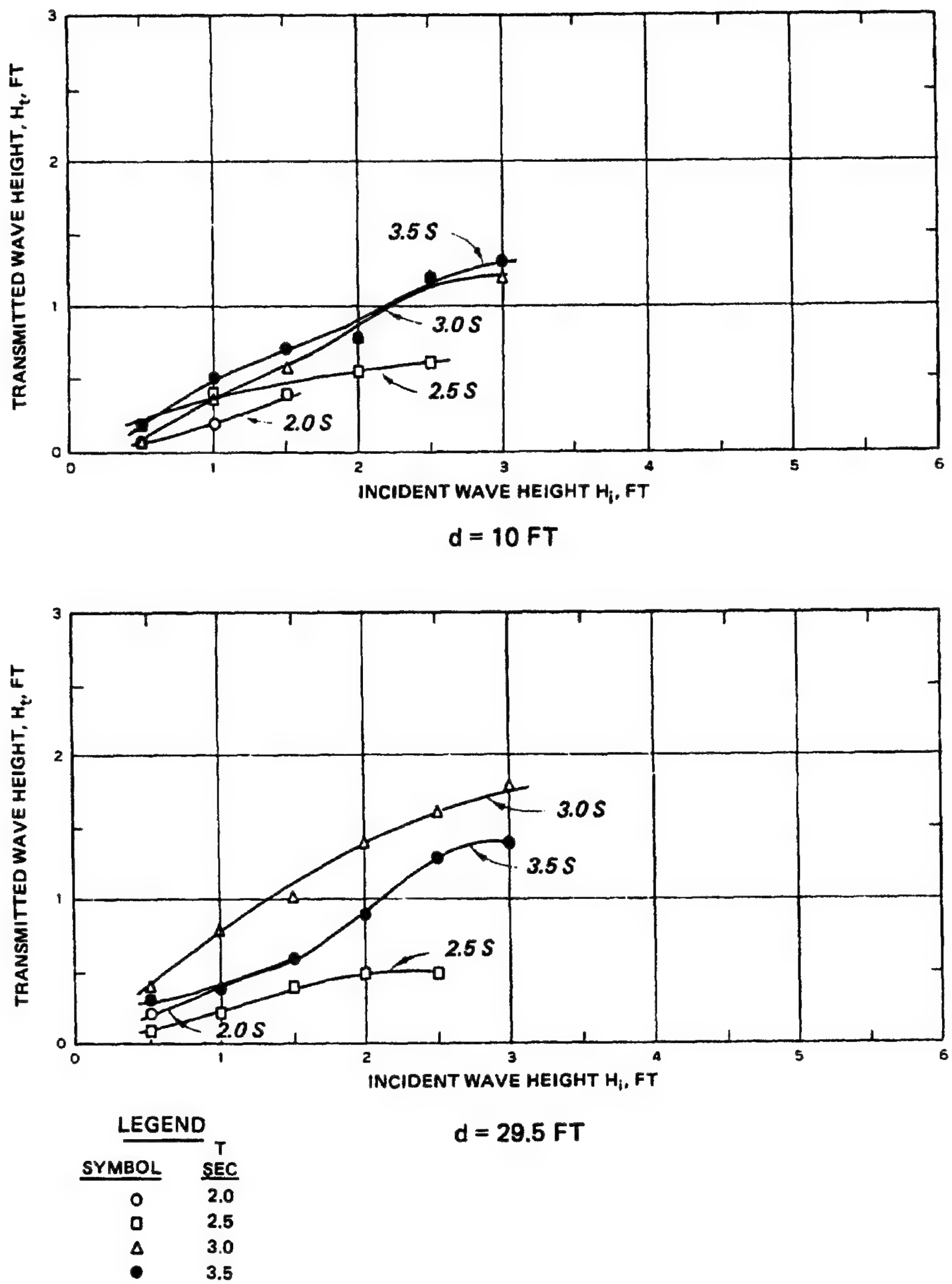
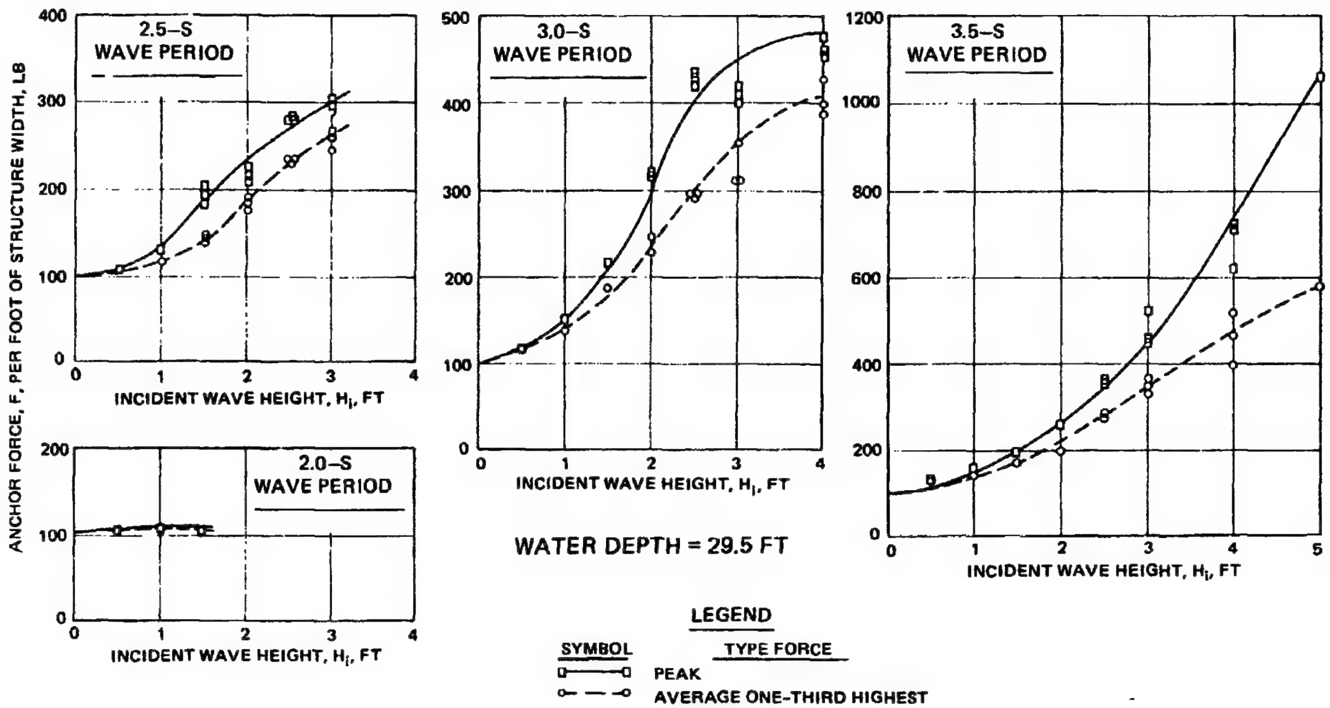


Figure 6-5 Wave transmission test results for the pile mooring system, catamaran-type floating breakwater, Oak Harbor, Washington



NOTE: TOTAL INITIAL FORCE ON THE SEA-SIDE ANCHORS WAS 100 LB PER FOOT OF STRUCTURE WIDTH

Figure 6-6. Anchor force test results for the chain mooring system, catamaran-type floating breakwater, Oak Harbor, Washington

incident wave height. Anchor chain force test results show that, although there was some scatter of the data points, definite trends were established from which the peak or average of the one-third highest force can be selected. The anchor force test results show, with the exception of the 2.0-second wave period, that the maximum peak anchor force is greater on the seaside anchors than on the harbor-side anchors. Considering the range of incident wave conditions at Oak Harbor, the maximum peak anchor force on the seaside was found to be about 300 pounds per linear foot of structure; the maximum peak anchor force on the harbor side was about 220 pounds per linear foot of structure.

(4) Anchor force tests with pile mooring system.

(a) During transmission tests on the pile mooring system, the forces exerted on the restraining piles in the direction of wave travel were measured. Thus, it was assumed that the forces applied by the module to the piles during testing would be in the plane of the still-water level. At the time of testing, the exact type of prototype pile to be used and its energy absorption characteristics had not been determined. Hence, it was assumed that if the forces on a pile with no deflection and absorption were known, it would be possible to determine with sufficient accuracy the forces on selected prototype piles with given deflection and absorption characteristics.

(b) Pile mooring force test results are presented in figure 6-7 as plots of the force on a pile per foot of structure width versus incident wave height. In each of the pile force plots, the solid line represents the maximum summation of forces per foot of structure width that simultaneously occurred on the model piles. The dashed lines represent the limits of the range of forces expected to occur on a pile due to the relative positions of the breakwater module and the pile. There is sufficient trend in the data to approximate the extreme forces exerted on a pile by the breakwater module under the given wave conditions. The maximum force on the seaside of the pile was found to be about 4,200 pounds per linear foot of structure width (2.5-second curve); the maximum force on the harbor side of the pile was about 4,600 pounds per linear foot of structure width. Before the pile mooring data from these tests are used for prototype design, the type of model mooring system used to obtain the pile force data should be noted and the resulting data adjusted in accordance with the deflection and absorption characteristics of the selected prototype piles.

(c) The Department of Public Works, State of Alaska, has developed a breakwater unit which consists of twin pontoons connected with cross pontoon sections. Modular construction was developed for ease of transportation to remote sites and for ease of assembly at the site.

(d) As shown in figure 6-8, a 21-foot-wide by 120-foot-long prototype structure was simulated. Tests were conducted with two different mooring arrangements: anchor chains crossed and uncrossed (figure 6-9). Wave periods

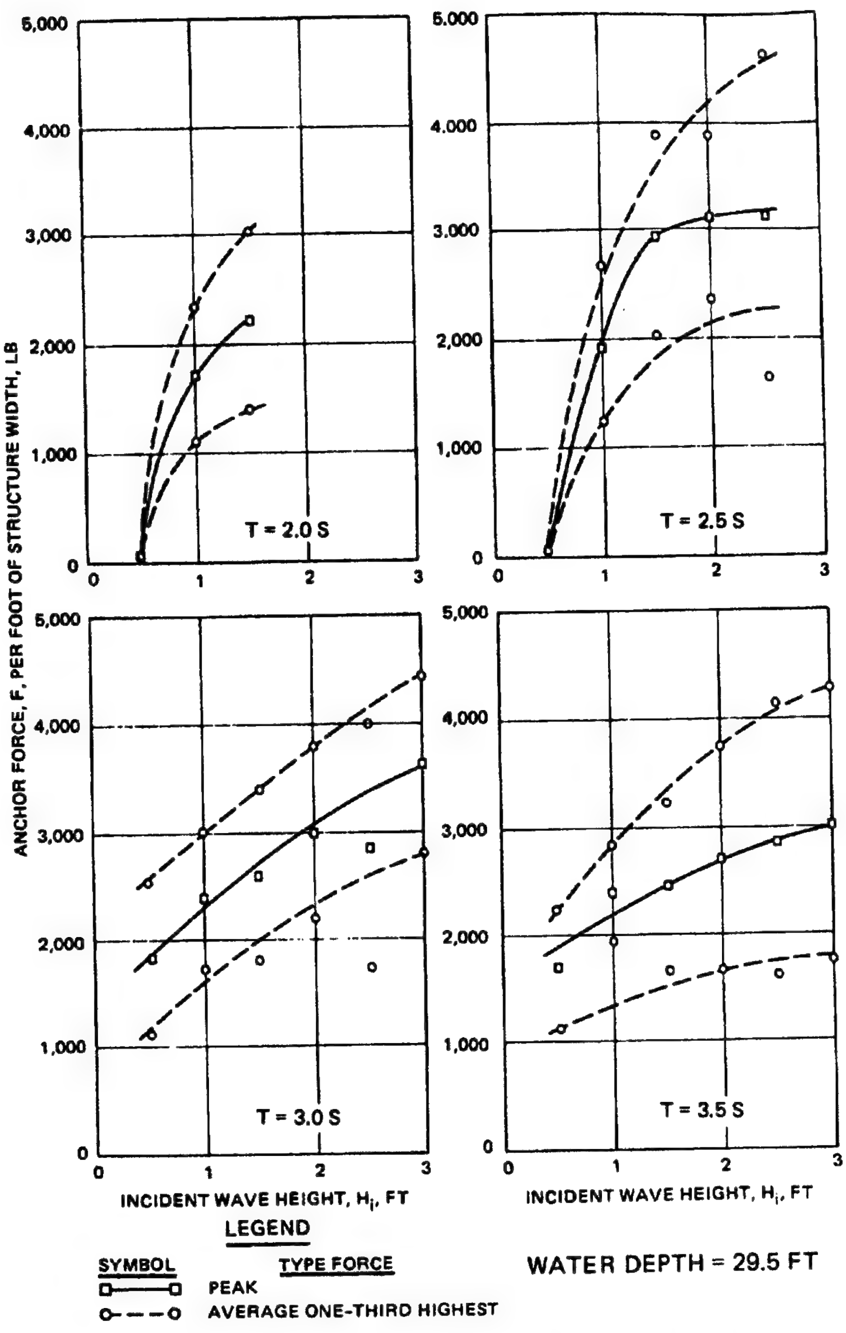


Figure 6-7. Anchor force test results for the pile mooring system, catamaran-type floating breakwater, Oak Harbor, Washington

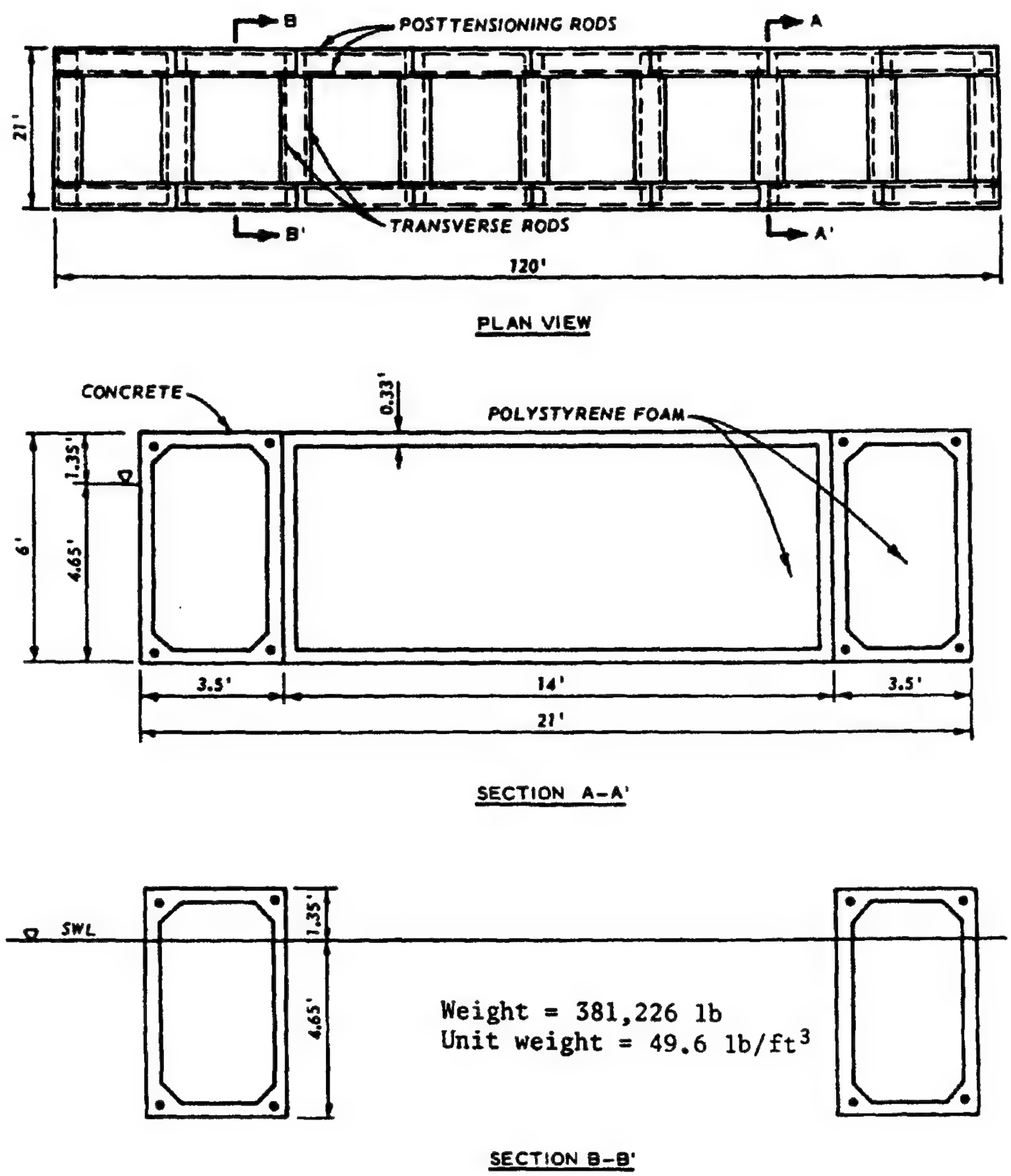
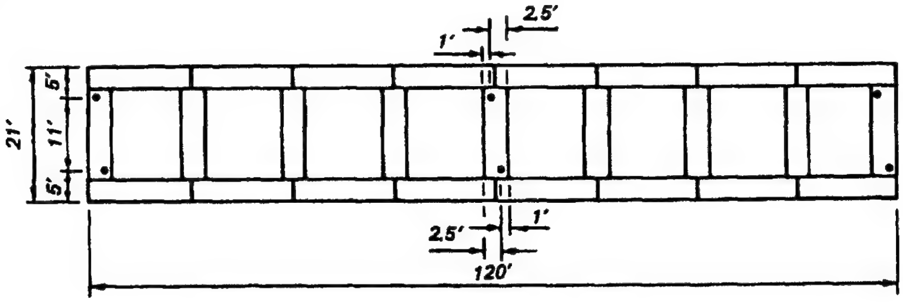
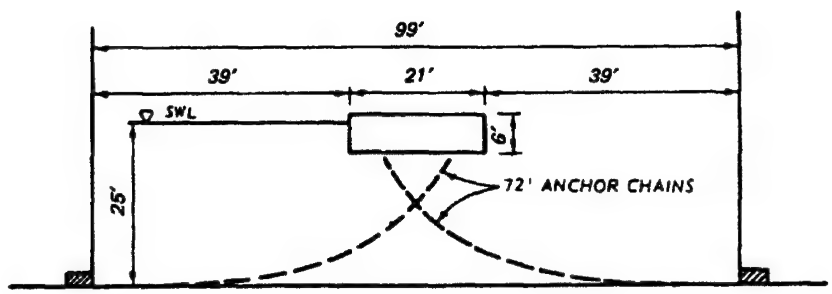


Figure 6-8. Details of Alaska-type floating breakwater evaluated in two-dimensional model tests for application at Olympia Harbor, Washington



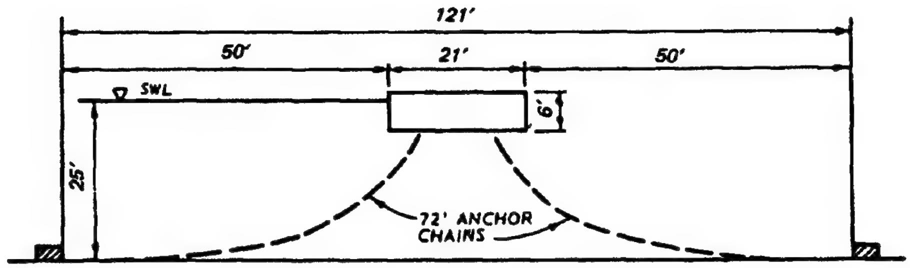
PLAN VIEW

• INDICATES ANCHOR POINT ON BOTTOM OF MODULE



END VIEW

ANCHOR CHAINS UNCROSSED



END VIEW

ANCHOR CHAINS CROSSED

Figure 6-9. Mooring chain arrangement for Alaska-type floating breakwater evaluated in two-dimensional model tests for application at Olympia Harbor, Washington

of 2.5, 3.0, 3.5, 4.0, and 4.5 seconds were tested in a water depth of 25 feet. These waves ranged in height from 1.5 to 3.5 feet.

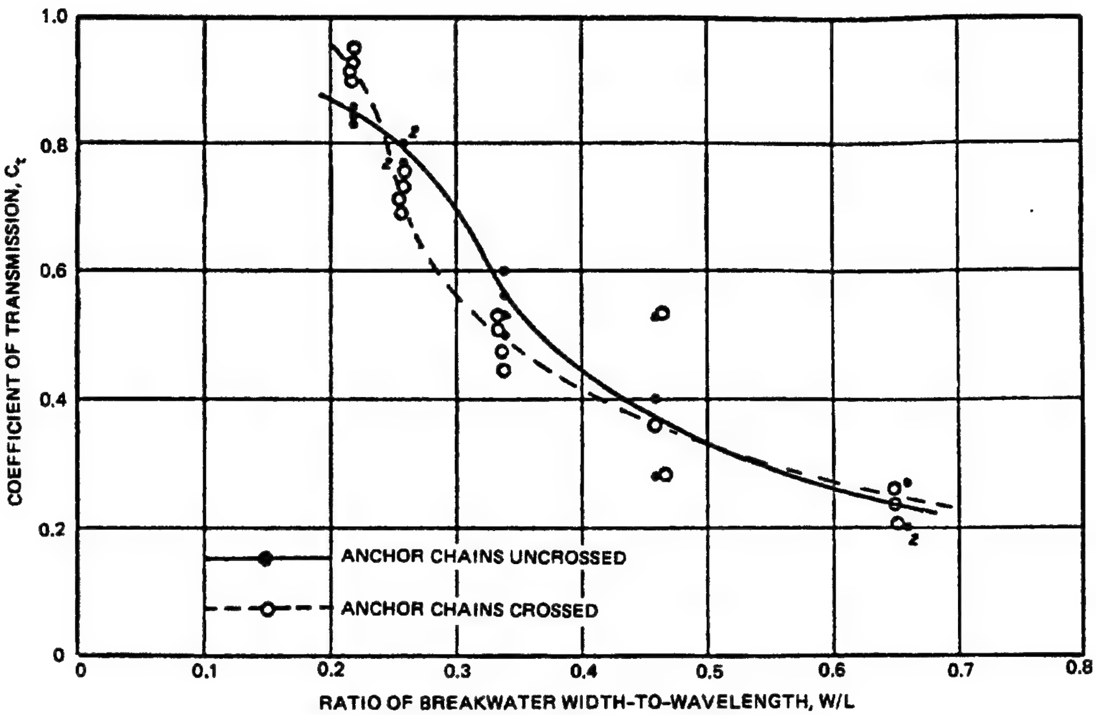
(e) Experimental results indicated that both anchoring arrangements gave almost identical values for the 2.5- and 3.0-second wave periods; however, the crossed arrangement yielded slightly lower transmitted wave heights for the 3.5- and 4.0-second wave periods. It appeared that the anchoring arrangement had a wave period-dependent effect on the amount of roll experienced by the structure and, hence, a wave period-dependent effect on transmitted wave heights. Observations of the Alaska-type floating breakwater under wave attack showed that for a 3-second wave period, an incident wave height of 1.5 feet produced a high degree of roll. However, as the incident wave height was increased to 2.0 and 2.5 feet, progressively larger amounts of water washed over the structure and dampened its rotation. The net result was that the transmitted wave heights observed for all three incident wave heights were nearly the same. The coefficients of transmission C_t versus relative breakwater width W/L resulting from these two-dimensional tests of the Alaska-type floating breakwater are presented in figure 6-10.

d. Construction Materials and Techniques. Construction materials for pontoon-type floating breakwaters must be resistant to ordinary solvents, particularly gasoline and petroleum. These structures are inevitably used as docking platforms, whether designed for this purpose or not. The materials and construction techniques appropriate for pontoon-type floating breakwaters are presented in items 1, 4, 95, 96, and 123.

(1) Concrete.

(a) Concrete provides the necessary mass and durability. The conclusion (item 116) that the displaced volume of water is far more important than breakwater shape has a ramification regarding the materials used for the construction of the breakwater; i.e., lightweight concrete should not be used. Maintaining mixing and placing standards is easier with regular concrete which has a long history of successful performance in saltwater. Durability and impermeability, the objectives for concrete used in a floating breakwater, are properties gained through good workmanship and the use of proper constituents. Chemical attack on concrete is hastened by sulfates and chlorides in sea water; in addition, chlorides promote corrosion of steel. High density and impermeability can be gained with a low water-to-cement ratio, a high cement content, proper air entrainment consolidation, and curing. Freezing and thawing resistance is gained from sound, proven aggregates and a dense mixture, generally, with a minimum design strength of 5,000 pounds per square inch. The concrete should conform to guidance given in EM 1110-2-2000.

(b) Prestressed concrete is used in a floating breakwater to keep all elements of the concrete in a compressive stress state. This prevents cracking of the concrete which would allow intrusion of water and salts. Prestressed concrete units also easily join together to form modules which may be assembled to produce a larger breakwater. Stressed steel is susceptible to



NOTE: NUMBERS BESIDE DATA POINTS INDICATE THAT THE NUMBER OF DATA POINTS EXCEEDS ONE

Figure 6-10. Coefficient of transmission versus relative breakwater width for Alaska-type floating breakwater evaluated in two-dimensional (2-d) model tests for application at Olympia Harbor, Washington

fatigue and corrosion from salt water, so the concrete should be sealed or otherwise protected. Because of hydrogen-embrittlement problems, prestressing steel should not be galvanized. It should be protected by cement grout or a commercial noncorrosive grease. Calcium chloride should never be used in prestressed concrete.

(2) Steel. The cyclic loading nature of a floating breakwater requires close scrutiny of the factors necessary to prevent fatigue and brittle fracture. Design stresses less than 20 percent of the yield stress will probably protect against crack propagation; this level is recommended for the critical areas of connections, anchor attachments, and other components. One of the most important considerations for the use of steel in a marine environment is its limited lifespan because of corrosion. Steel should preferably be hot-dip galvanized after all fabrication and welding. Alternatively, there are many proprietary coatings on the market, the best of which appears to be a coal-tar epoxy amine type applied over a zinc-rich primer on a sandblasted surface.

All stainless steels exhibit some susceptibility to seawater corrosion. All accessories embedded in the concrete pontoon should be noncorrosive materials which will not promote galvanic action; galvanized steel and stainless steel have been used successfully.

e. Flotation Materials. Floating breakwaters can be filled with polystyrene or other flotation materials to insure buoyancy. Certain compartments can be left open for weighting of the structure to allow even flotation characteristics (this technique is much simpler than adding flotation to a breakwater which was otherwise overweighted). The method of providing flotation should allow for punctures and leakage by including a redundancy in the form of bulkheads or simply the interconnection of all components. The flotation material must be resistant to, or protected from, impact and deterioration. Polystyrene foam is both gasoline and solvent resistant; its equivalent or better should be specified for most uses in floating breakwaters.

f. Module Connections. All hardware and mechanical connections necessary to join modules of a floating breakwater should be carefully sized to exceed the strength of the anchor lines in retaining the structure. The connections (shackles, clevises, swivels, bolts, pins, etc.) usually experience the greatest wear and motion and should have secondary methods of loss prevention such as cotter pins or double nuts. Custom designed and fabricated connecting devices have been found to be the best and most economical, but compatible materials can be used to lessen galvanic action.

g. Anchoring Systems.

(1) Anchors. The type of anchoring system designed for a particular location depends to a large extent on the type of bottom material at that specific site. Conventional ship-type anchors may be available in the 6,000- to 8 000-pound range, but their holding power under actual site conditions has not been field tested. Pile anchors are quite effective if penetration is sufficient to develop adequate shear and bending strength of the pile. Many bottom locations have at least a few feet of soft or otherwise favorable material for anchor placement. A concrete mass anchor is only capable of developing a resistance to movement of about one-half its submerged weight if the ground is firm enough to resist settling. Both concrete mass anchors and pile anchors for floating breakwaters are discussed in item 51.

(2) Anchor Lines. Acceptable materials for a floating breakwater anchoring system are synthetic fibers, chain, or wire rope. Design comparisons should consider cost, size, working strength, and elasticity.

(a) Chain. Chain is available in many grades and types of materials. Chain derives its energy absorption capabilities from the components of weight and the resultant catenary effect which effectively functions as a spring. Connection is easily provided at any point on its length. Anchor chains which are not galvanized should be designed oversized to allow for corrosion. This oversizing is beneficial because the weight gained yields a deeper catenary

curve and more absorption capability because of the spring effect. Mooring chains and joints always experience repeated loading, causing a decrease in strength from fatigue and a loss in chain diameter through abrasion and corrosion.

(b) Synthetic fibers. Nylon, Dacron, or polypropylene synthetic lines each have unique characteristics to be considered, but nylon is more practical because of its energy-absorbing nature (the fundamental purpose of the floating breakwater system). The size of nylon lines is important because the elongation and resultant lateral movement of the floating breakwater must be kept within reasonable limits. The recommended factor of safety for synthetic lines is 4 to 5. Pertinent to the design of a floating breakwater is the availability of sufficient reserve strength for the rare storm which would greatly exceed the normal working loads. Prototype observations indicate that it would be a rare condition if the entire length of a floating breakwater were loaded uniformly at a particular time. It is more probable that only a small percentage of the total number of anchor lines will be fully loaded at any given time.

h. Advantages and Disadvantages of Pontoon Breakwaters.

(1) Advantages.

(a) Fifty-year design life.

(b) Simple shape to build.

(c) Proven performance.

(d) Effective in moderate wave climate (wider range of application than scrap-tire breakwaters).

(e) Unit will allow pedestrian access for fishing and temporary boat mooring.

(2) Disadvantages.

(a) High cost compared with scrap-tire type.

(b) Maintenance, if damaged, may require towing to drydock.

(c) Requires large connector forces.

6-5. Scrap-Tire Floating Breakwaters.

a. General. Systematic investigations of the use of scrap tires as floating breakwaters have been limited to the past 20 years. Stitt and Noble developed and patented the "Wave-Maze," a geometric assembly configuration (item 122). The Goodyear Tire and Rubber Company has investigated the use of

modular building-block elements formed by securing together bundles of tightly interlocked scrap tires with high-strength rope or cable, but the company has not patented nor commercially used scrap tires in this form (item 17). The information from this research has been made available for public use. Kowalski tested a simple mat-type floating breakwater of scrap automobile tires, constructed in various layers of mats fastened together (item 82). Harms experimentally investigated a concept known as the "wave-guard" (now the "pipe-tire" structure) which differed from both the Wave-Maze and the Goodyear concept (item 58). Structural components of massive logs (telephone poles, concrete beams, etc.) were utilized, with the scrap tires being threaded onto the poles which were in turn connected with conveyor belting.

b. Wave-Maze Floating Breakwater.

(1) The patented Wave-Maze scrap-tire floating breakwater (item 122) was subsequently investigated for performance effectiveness (items 79, 101, and 102). The basic component of the breakwater is used truck tires, some of which are filled with flotation material such as polystyrene or polyurethane. The construction consists of both a top horizontal layer and a bottom horizontal layer of truck tires bolted to a center element of vertical tires arranged in a triangular pattern (figure 6-11). Each line of tires in plan view is approximately 4.5 feet wide. The breakwater should be constructed so that its total width is at least one-half of the length of wave to be attenuated. If wave heights are greater than about 4 feet, additional tiers of tires should be added so that the depth of the wave-maze exceeds the wave height to be attenuated. Truck tires were recommended instead of automobile tires because the extra sidewall piles in the casing help reinforce the connections. At least two layers of reinforcement material (i.e., sections of tire casings or conveyor belting) should be added inside the tires at each bolted joint. Hot-dip galvanized bolts and washers should be used for all connections in saltwater environments.

(2) The Wave-Maze physical model (item 79) was constructed of 6-inch-diameter tires assembled in the same fashion as in the prototype, with one exception: the method of fastening the tires together. In the prototype, the tires were fastened together by bolts; because of the size of tires in the physical model, wire connections were used instead. The precise effects of this connection method are unknown, but it is believed to allow relatively consistent comparable flexing of the assembly.

(3) Analysis of the test data indicated that the relative height to which the breakwater extends above still water does not seem to affect the wave reflection coefficient C_r or the wave transmission coefficient C_t. This was due to the high flexibility of the breakwater which moved extensively as if it were a part of the water surface. At the same time, a large increase in the relative penetration into the fluid (i.e., relative submergence y/d) resulted in only a small decrease in the coefficient of wave transmission. These data are presented in figure 6-12 which shows the effect of initial wave

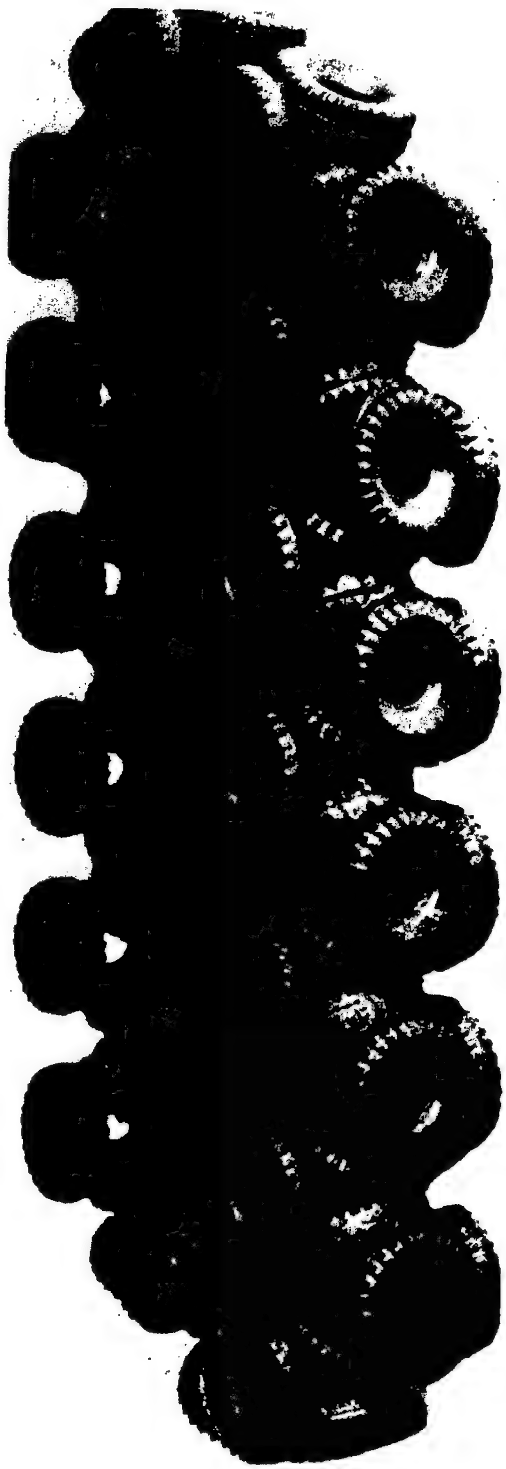


Figure 6-11. Wave-Maze floating breakwater model structure

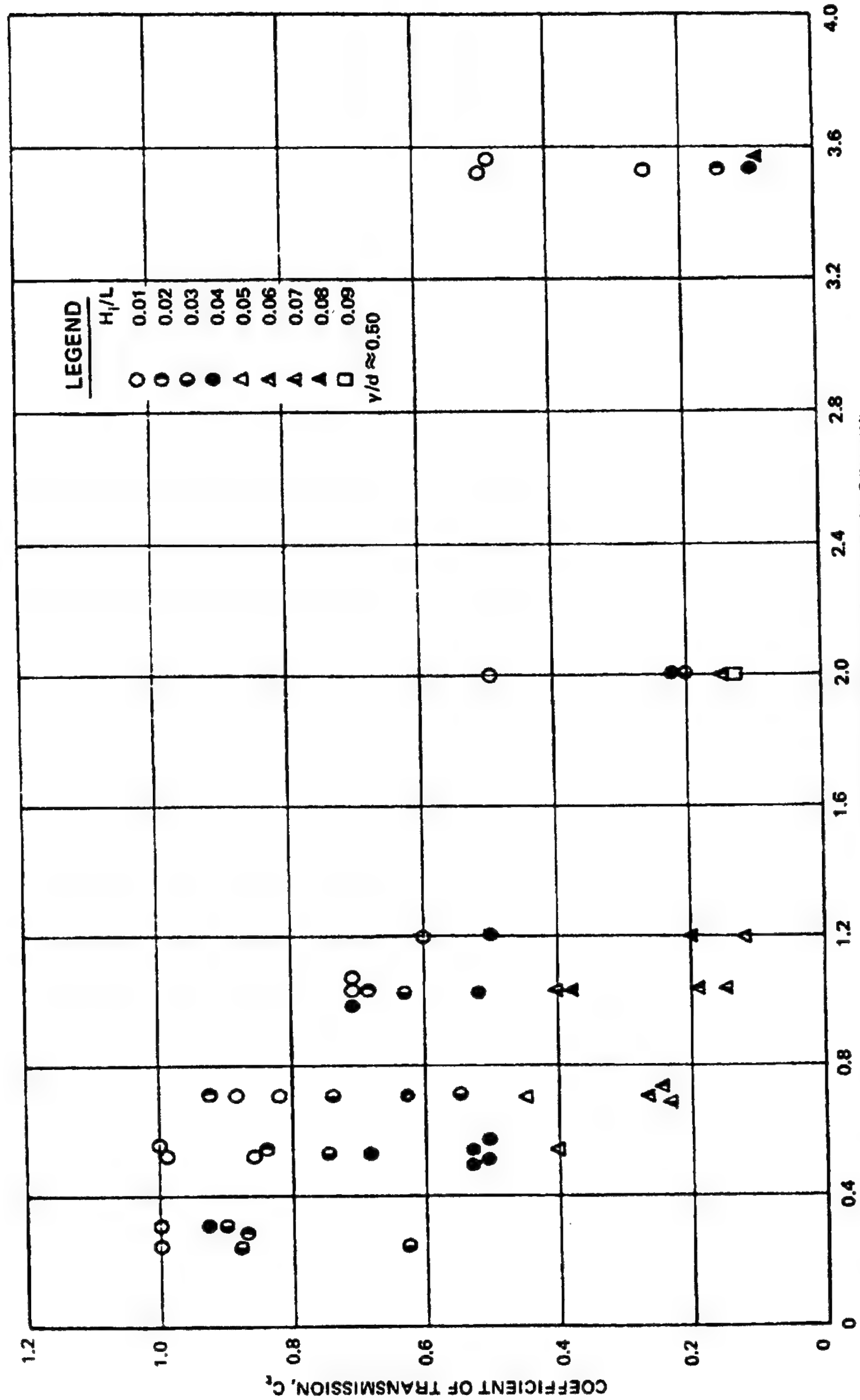


Figure 6-12. Effect of wave steepness, H/L , and relative breakwater width, W/L , on coefficient of transmission, C_t , for the Wave-Maze floating breakwater

steepness H_i/L on the coefficient of transmission C_t and in figure 6-13 which displays the effect of relative submergence y/d on the transmission coefficient C_t. Relative submergence is defined as the ratio of draft (y) to water depth (d).

c. Goodyear Floating Breakwater.

(1) The Goodyear floating breakwater concept uses a modular building-block design. The section is constructed of units of relatively few tires secured together to form a small, easily assembled, portable building unit which serves as the basic element for constructing the large structure. The simple construction procedure is accomplished by securing 18 individual tires together to form a 7- by 6.5- by 2.5-foot tightly interlocked bundle of scrap tires (item 18). The basic method of constructing the tire modules is to stack the tires flat, but vertically, in a 3-2-3-2-3-2-3 combination (figure 6-14), constantly interweaving the tying material. The increasing weight of the tire stack and the physical compression of the tires during assembly will compress the tires enough to allow easy fastening of the tying material and formation of a tightly secured bundle. After construction, the modules are easily transportable for assembly at the project location.

(2) Of the interlocking materials investigated by Goodyear Tire and Rubber Company as of 1976, specially manufactured, unwelded open-link chain, 1/2 inch in diameter, proved to be best suited for the construction of scrap-tire floating breakwaters. The open-link chain has adequate strength, is easily handled, and has a long life expectancy in seawater. It is also easily interconnected with the use of simple hand tools. The use of dissimilar metals should always be avoided in a marine environment.

(3) Prototype-scale mooring load and transmission tests for the Goodyear floating tire breakwater concept are reported in items 51, 52, and 53. Tests were conducted in the Coastal Engineering Research Center's (CERC) large wave tank which is 20 feet deep, 15 feet wide, and 635 feet long. Waves of constant period and height were produced by a piston-type generator.

(4) Two floating tire breakwaters (one containing 8 Goodyear modules, the other 12 modules) were tested. The breakwaters included modules constructed with 14- and 15-inch automobile tires, two modules wide across the tank and four or six modules along the tank (the width of the breakwater in the direction of wave advance). Each section was tested using wave conditions commonly found on a sheltered body of water such as a reservoir or bay. A total of 165 combinations of wave period, wave height, structure width, and water depth were tested. Wave periods ranged from 2.64 to 8.25 seconds. Wave heights varied from 20 to 140 centimeters (0.6 to 4.5 feet) at water depths of 2 and 4 meters (6.5 and 13 feet). Each combination of wave height, wave period, water depth, and structure width was tested for

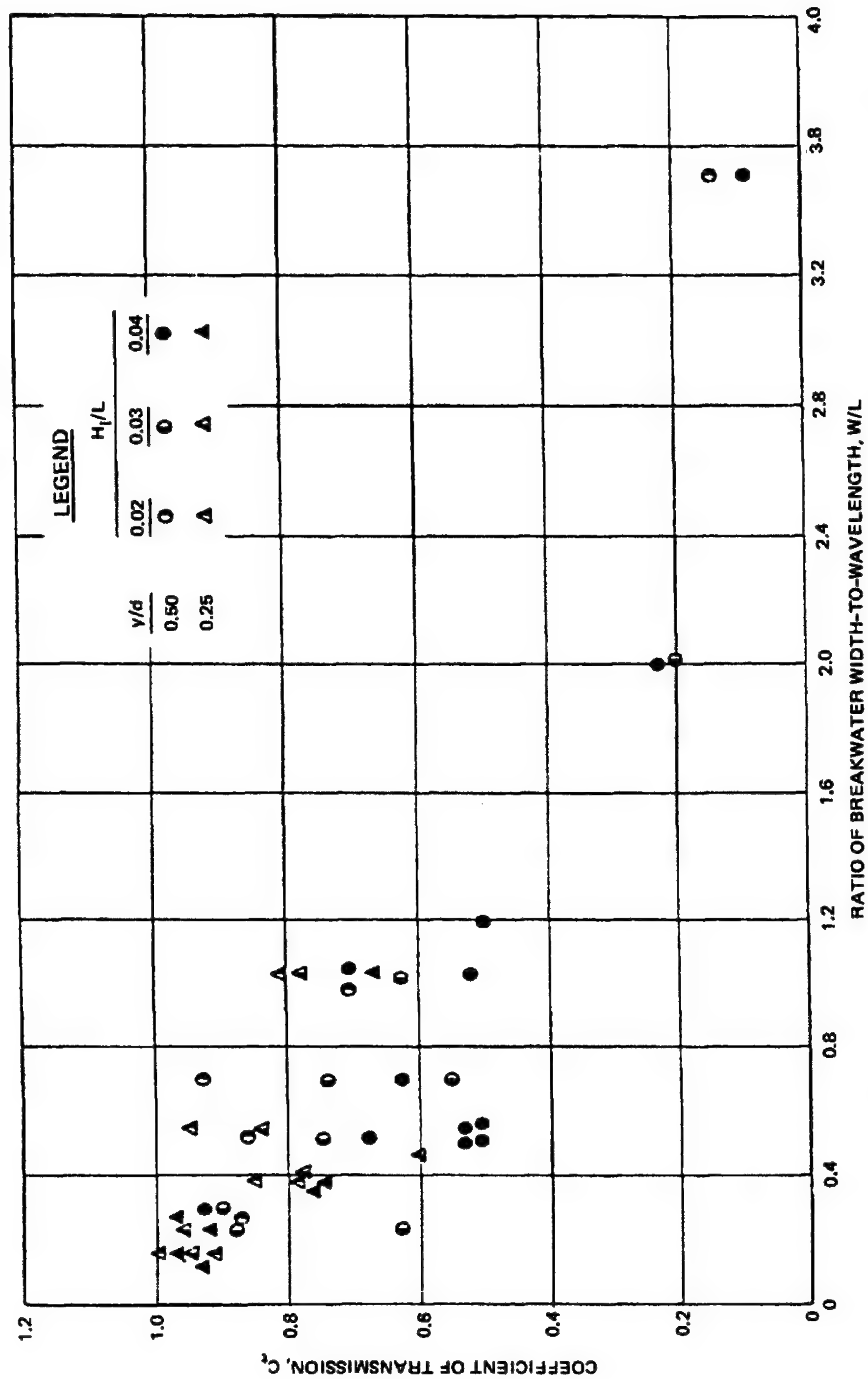


Figure 6-13. Effect of relative submergence, y/d , and relative breakwater width, W/L , of coefficient of transmission, C_t , for the Wave-Maze floating breakwater

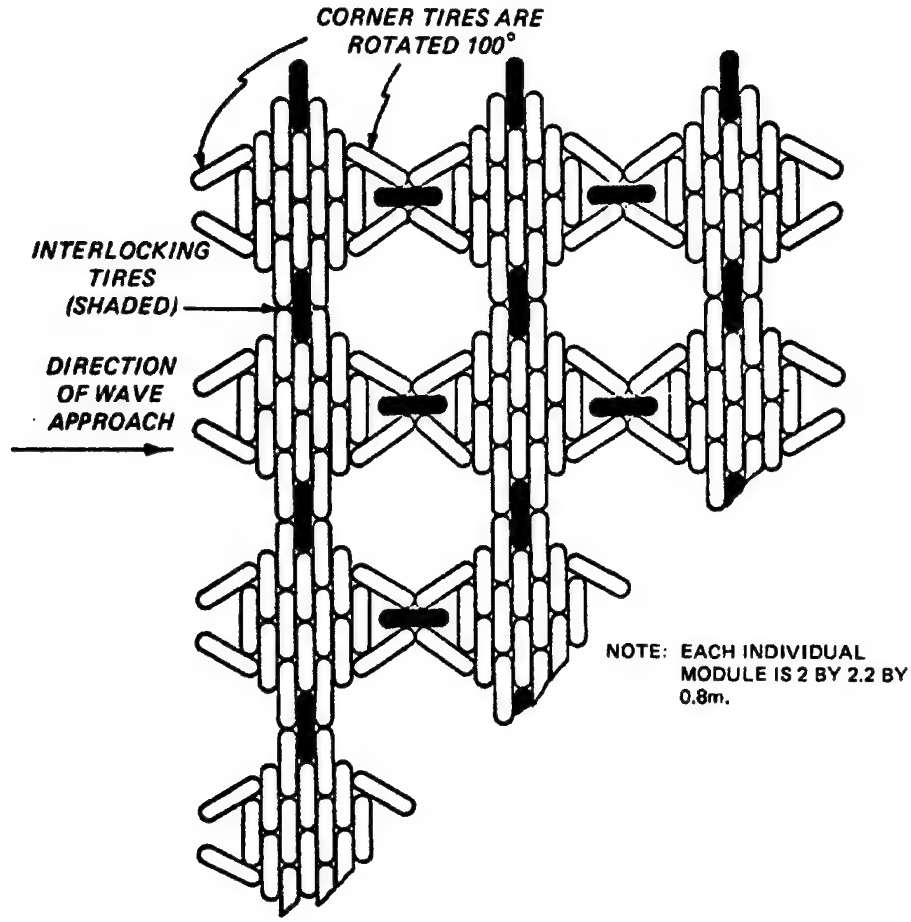


Figure 6-14. Assembly of modules in a section of the Goodyear Tire and Rubber Company scrap-tire floating breakwater

5 minutes, which allowed sufficient time to determine the pertinent forces and wave heights.

(5) The transmission coefficient C_t versus the breakwater width-to-wavelength ratio W/L is shown in figure 6-15. This graph effectively constitutes a design curve, as all data are shown and the range of incident wave heights is indicated by the legend symbols. (Designers should not extrapolate beyond W/L = 1.40 or apply these data to breakwaters with a width of more than 12 modules.) Generally, the data show that as W/L increases, the transmission coefficient C_t decreases; also, for the same value of W/L , as the incident wave height increases, the transmission coefficient decreases slightly. There is considerable scatter in the data for W/L values less

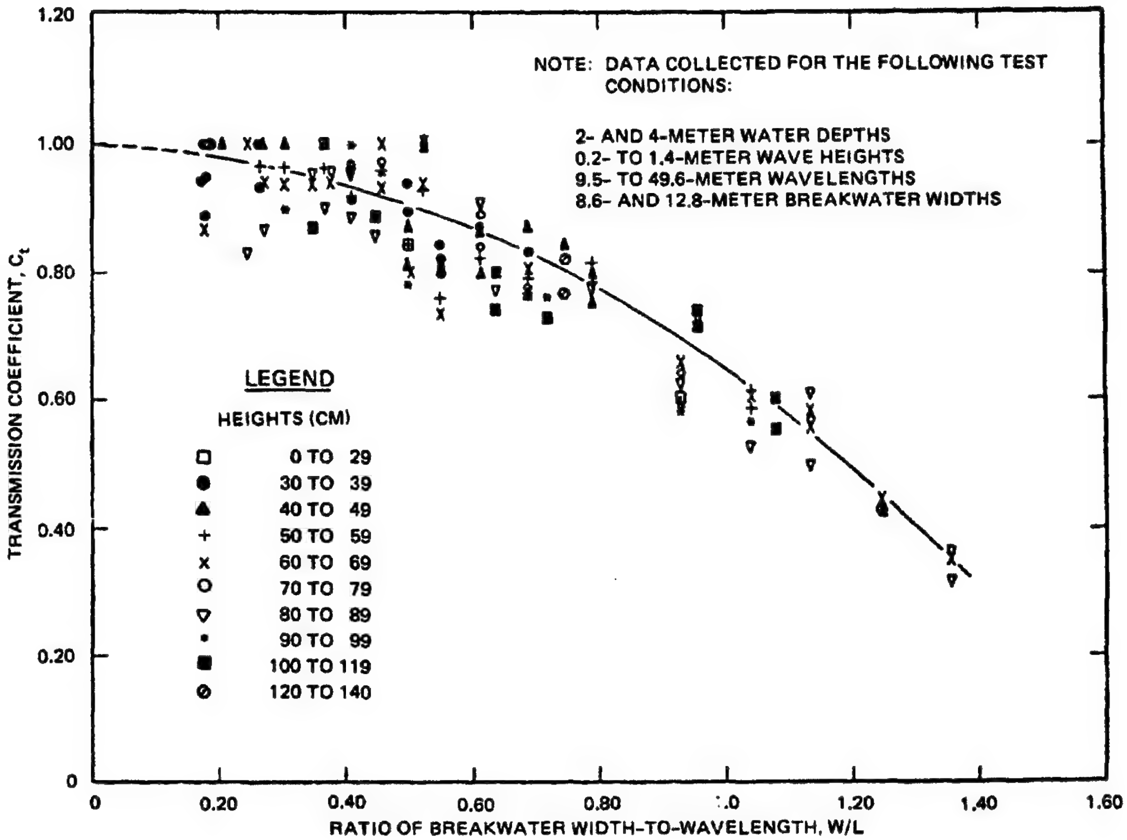


Figure 6-15. Design curve for determining width, W , of the Goodyear Tire and Rubber Company floating breakwater concept for various wave lengths, L , and allowable transmission coefficients, C_t

than 0.40 because the incident wave height was usually small and was only 2 to 4 centimeters (0.05 to 0.13 foot) greater than the transmitted height; thus, a small change in the measured transmitted height caused a large change in the value calculated for the transmission coefficient C_t . A comparison of the data at 2- and 4-meter water depths shows that for the conditions tested the water depth does not appear to influence the transmission coefficient. This observation is contrary to the expectation that as more of the water depth is taken up by the breakwater section, the wave attenuation should increase.

(6) During testing of the prototype-scale floating tire breakwater at CERC, peak and average mooring forces also were measured. Results of these tests (figure 6-16) show that the peak forces are not significantly greater

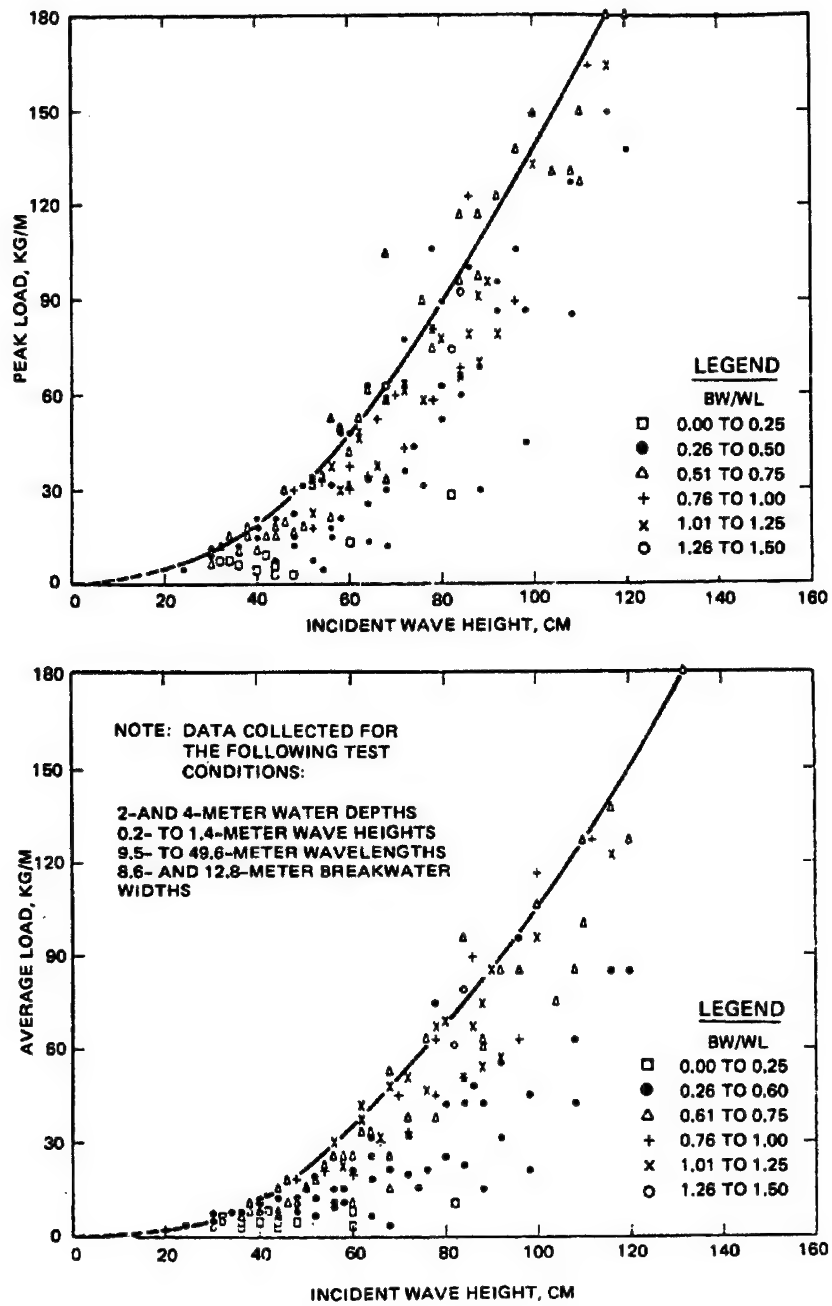


Figure 6-16. Design curves for predicting peak and average mooring forces for the Goodyear floating breakwater concept

than the average forces. No strong wave period dependency was discerned in the data for either peak or average mooring forces.

(7) Since the peak force test represented a situation in which the breakwater was initially at rest and then subjected to monochromatic waves, the maximum force calculated using the peakload curve would probably be somewhat larger than the peakload that would occur in a train of irregular waves. Therefore, a conservative force prediction for the Goodyear Tire and Rubber Company scrap-tire floating breakwater concept would be to obtain the mooring force load based on the peakload curve.

d. Wave-Guard Floating Breakwater.

(1) Harms developed and tested a scrap-tire floating breakwater which differs principally from other concepts in terms of tire arrangement (spatial tire density) and rigidity (item 58). This concept, called the "wave-guard" (also referred to as the "pipe-tire" structure), was experimentally tested at model scale. The structural component of the wave-guard is massive logs (telephone poles, steel beams, reinforced concrete beams, etc.). Strips of conveyor belting are used to connect one beam to another and to thread the scrap tires. The tire strings are closely spaced (figure 6-17) so that the spatial density (number of tires per unit volume of breakwater) is relatively high, which results in a tightly packed structure. Thus, a structure significantly smaller in planform area is required to produce the same wave attenuation.

(2) The wave-guard was tested for the same wave conditions as the Goodyear breakwater. Results of these tests show that the wave-guard offers a significantly greater degree of wave attenuation than the Goodyear concept (figure 6-18). This increased performance is probably attributable to the greater rigidity of the wave-guard and to the fact that it is much less porous than the Goodyear structure.

(3) In the wave-guard tests, a mooring line with a three-tire mooring damper was installed. This arrangement allowed the mooring connection at the breakwater end to be made directly to the massive beams, rather than to the more flexible but weaker tire connections, without incurring excessively high peak mooring loads. Since full-scale tires are stiffer than the one-eighth-scale-model tires tested, it was recommended that at least five tires be used in the full-scale mooring damper. Structural failures of scrap-tire floating breakwaters often occur because of stress concentrations near the mooring connection.

(4) Design curves of the mooring force parameter $F/\gamma W^2$ were developed for the wave-guard and compared with the corresponding curves of the Goodyear Tire and Rubber Company's concept. Because of the greater wave attenuation capacity of the wave-guard, a larger amount of wave energy is dissipated by this structure; hence, the forces existing on the moorings are accordingly increased. These force-parameter comparisons are presented in figure 6-19.

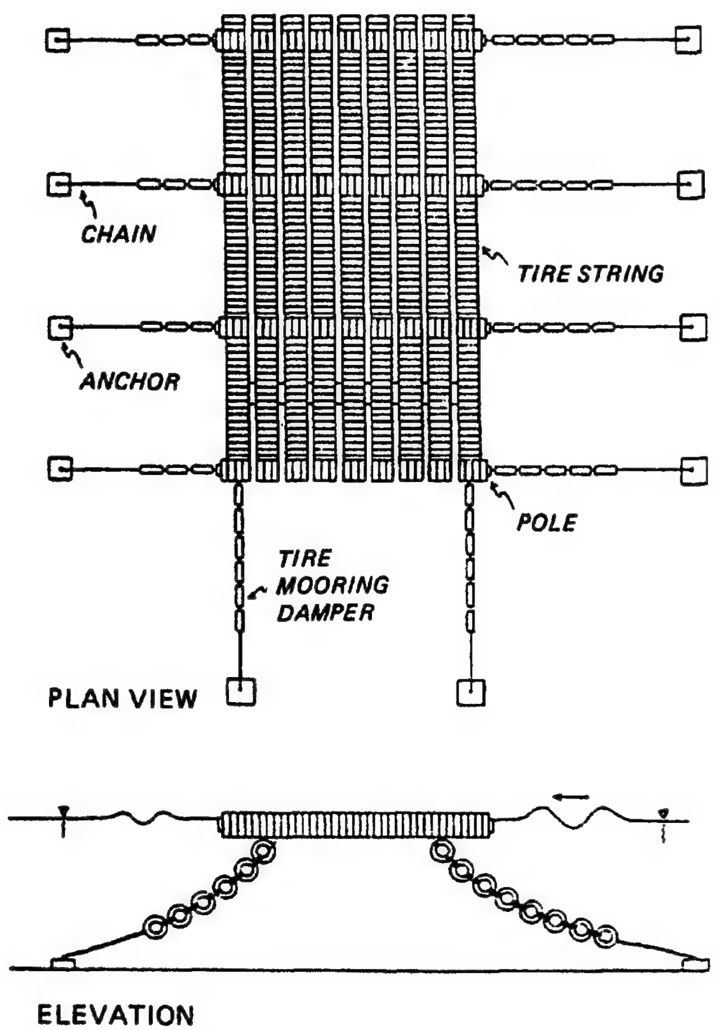


Figure 6-17. Schematic of wave-guard scrap-tire floating breakwater concept

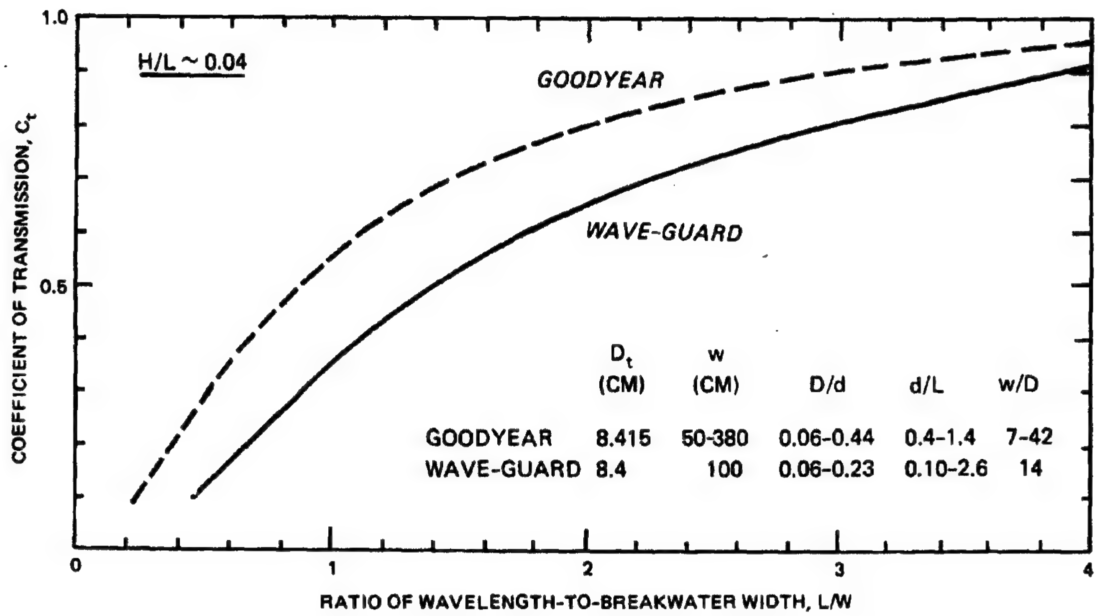


Figure 6-18. Comparison of wave transmission coefficient C_t for wave-guard and Goodyear floating breakwater concepts

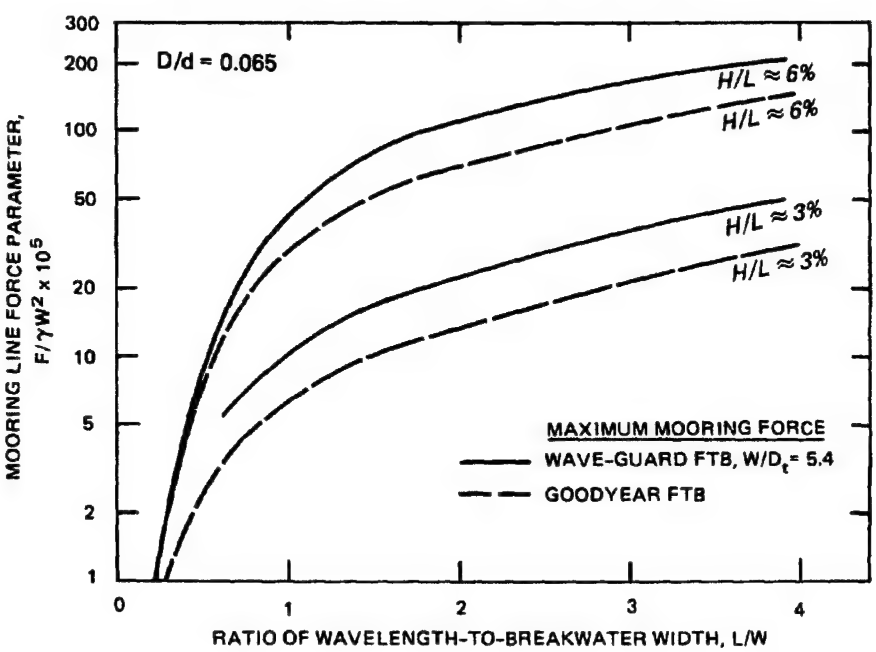


Figure 6-19. Comparison of force parameter $F/\gamma W^2$ for wave-guard and Goodyear floating break-water concepts

e. Construction Considerations. Procedures for assembly of unit modules to fabricate the Goodyear concept are provided in items 17, 18, 49, 83, and 118. Guidance on construction of the wave-guard concept is provided by item 58.

(1) Tire assembly.

(a) The design of the Wave-Maze and wave-guard is so complex that assembly will probably be required on-site. The Goodyear concept, in which relatively few tires are secured together to form a portable building-block for larger structures, can be transported from the assembly site to the breakwater location. Connecting materials for assembling the tires of the Goodyear concept include heavy steel chain or conveyor belting materials. The Wave-Maze is constructed by bolting together tire sidewalls, using pieces of conveyor belting as reinforcement washers; hence, the heavier truck tires are recommended in this concept.

(b) In-situ saltwater tests to evaluate the reliability of 12 different potential materials for connectors have been conducted (items 48 and 118). The binding material recommended above all those tested is conveyor belt edging material (a scrap product resulting from the trimming of new conveyor belts). This material demonstrated ultimate tensile strength on the order of 9,500 pounds per square inch and is available from several manufacturers. Minimum recommended belt dimensions are 2 inches wide by 0.375 inch thick, with three or more nylon plies. This material can be easily cut with a band or hacksaw, and holes can be punched singly or with a multiple punch. Conveyor belting is virtually inert in the marine environment. The use of nylon bolts, nuts, and washers as a means of fastening the belting together is recommended; heavy steel chain is recommended as a secondary choice. Materials definitely NOT recommended for assembly of the units include nylon lines (poor abrasion resistance, knot-loosening, and ultraviolet degradation) and metallic-wire rope (inherent corrosive problems, metal fatigue, and cutting action of the rope on the tire body).

(2) Foaming for buoyancy. Air trapped in the tire crowns provides sufficient buoyancy to keep a floating tire breakwater afloat for a short period of time. However, to ensure that the structure remains in a position to provide protection for up to the estimated 10-year life, and to compensate for the added weight of marine growth, supplemental flotation should be added in every tire. A technique for onsite foaming of scrap tires that can be easily handled by one or two people is described in item 17. This technique uses simple, flat plate molds to hold expanding urethane foam inside the tire. The foam is a two-component pourable mixture of a 1:1 ratio by weight which can be mixed easily by an electric drill-type mixer. The liquid foam can then be poured into the tires where it expands and cures in about 15 minutes. It may be necessary to vent the top half of the tire if trapped air voids occur under the sidewall areas. This is easily accomplished by drilling holes through the upper part of the tire to allow air to escape as the foam rises. Other types of flotation materials, such as molded polyethylene floats or 1/2-gallon

plastic bottles inserted into the tires, have also been used. Completely uniform flotation will facilitate interconnecting the units in water, and the independent flotation of each unit allows the interconnecting hardware to be used with maximum efficiency.

(3) Mooring systems.

(a) The type of line or chain used to moor a floating tire breakwater is important from the standpoint that it must be strong enough and resilient enough to withstand peak forces and fatigue. Local experience in mooring large ships has been used as a guide, and past studies have indicated that the vertical load on the anchor should be minimized. The mooring line should have a minimum length of approximately eight times the maximum expected water depth, and the anchor should be positioned seven times the maximum water depth from the breakwater (item 51). During storm conditions, local seas have to lift the mooring line off the bottom before forces are applied directly to drag the anchor; hence, many builders have used chain (either galvanized steel or wrought iron) rather than other materials in the mooring system. Wire cable has occasionally been used, but cable is subject to both axial fatigue and corrosion weakening. Chain moorings should be attached to the breakwater in a manner that distributes the load between two or more modules. This can be accomplished by attaching a short bridle to the outer tires of the module and then attaching the mooring chain to the bridle.

(b) Because of its unique construction aspects, the recommended mooring line for the wave guard (item 58) consists of a tire mooring damper located at the breakwater end of the mooring line, plus an anchor chain near the bottom (refer to figure 6-17). The tire mooring damper should consist of at least five tires in series. The mooring line should be fastened to the poles or piling through two tires that are located approximately 10 tires from the end.

f. Advantages and Disadvantages of Scrap-Tire Floating Breakwaters.

(1) Advantages.

(a) The cost of the scrap-tire breakwater is low.

(b) It is easily removed and beached for maintenance or to prevent ice damage.

(c) It can be constructed with unskilled labor and minimal equipment.

(d) It has relatively low anchor loads.

(e) It produces low reflected wave heights.

(2) Disadvantages.

(a) In order to ensure flotation, foam is usually needed for extra buoyancy and regular maintenance is needed to control marine growth.

(b) The design life appears to be only 10 to 20 years.

(c) It is effective only in mild wave climates (the upper limit of applicability is about 3-second, 3-foot waves).

(d) The Goodyear concept tends to entrap litter.

(e) Marine growth.

(f) Appearance.

6-6. Models. Models may be needed to predict wave transmission and anchor loads. Mathematical models are suitable for preliminary design; however, physical hydraulic models may be needed for final design optimization. Two-dimensional flume tests are used to determine wave transmission and anchor loads. Three-dimensional models are used to determine wave heights in the area of protection due to transmission through and diffraction around the breakwater.

6-7. Prototype Tests. In 1981, the US Army Corps of Engineers initiated a prototype test program to establish design criteria for floating breakwater applications in semiprotected coastal waters, lakes, and reservoirs. The tests were designed to obtain field information on construction methods and materials, connector systems, and maintenance problems and to measure wave transmission characteristics, anchor loads, and structural forces. The structures that were built are of two types: a concrete box design and a pipe-tire mat design. The 150-foot-long concrete breakwater was composed of two 75-foot-long units, each 16 feet wide and 5 feet deep (draft of 3.5 feet). The pipe-tire breakwater was composed of nine 16-inch-diameter steel pipes and 1,650 truck tires fastened together with conveyor belting to form a structure that was 45 feet wide and 100 feet long. The following conclusions based on prototype test results are summarized from item 13.

a. Both breakwaters provide satisfactory protection (transmitted wave height of 1 foot or less) for waves up to 3 feet high.

b. Most of the urethane foam flotation in the crowns of the tires of the pipe-tire breakwater remained securely intact and in place throughout the test. The durability of the foam was enhanced by the physical protection provided by the very stiff sidewalls of the truck tires. If more flexible automobile tires were used, the foam probably would be more vulnerable to damage. In one year, the average foam weight increased 250 percent due to the absorption of water. This absorption combined with underfilling of tires during the original construction could have led eventually to buoyancy problems. The long-term water absorption rate of foam flotation remains a concern, and should be taken into account when flotation requirements are

being calculated. The pipe-tire breakwater original design flotation requirement of 75 pounds positive buoyancy for tires, other than those on the beamwise pipes, is probably not overly conservative for long-term use.

c. Although a number of the bolted connections had one or two broken bolts, none of the connections failed, and binding the tires of the pipe-tire breakwater with loops of conveyor belting, and fastening the loops together with nylon bolts appears to produce a strong durable structure.

d. The 16-inch-diameter pipe for the pipe-tire breakwater should be used in standard lengths to avoid welding. If welding is required, all welds should be carefully inspected.

e. Construction cost of the prototype tests' 150-foot-long concrete breakwater was approximately $2,600 per lineal foot (1981). In 1983, construction of a 1,500-foot-long breakwater of similar design (anchored in a similar depth) cost $1,500 per lineal foot indicating a considerable cost reduction for larger projects.

f. Construction cost of the prototype tests 100-foot by 45-foot pipe-tire breakwater was $1,600 per lineal foot (1981) including anchors. Based on experience with the concrete floating breakwater, a large project is expected to cost considerably less.

g. When considering either structure the method of energy dissipation should be considered. The concrete breakwater reflects the waves causing a "rougher" environment in front of the breakwater; whereas, the tire breakwater used friction which cuts down on the wave reflection.

6-8. Maintenance. All anchor lines and intermodule connections on floating breakwaters should be periodically inspected for wear and abrasion and repaired or replaced as needed. Marine growth should be removed if it becomes extensive enough to significantly affect the flotation height of the structure. Guard rails and walking surfaces should be kept in safe condition if pedestrian access is provided. Concrete structures should be inspected for cracking and sealed as needed to prevent intrusions of water.

6-9. Rehabilitation. Floating structures that have sustained major damage from storms, boat collisions, or other events may require rehabilitation. The modular construction techniques employed for tire and concrete floating breakwaters facilitate replacement of sections of the structure. Replacement of anchor lines may be required if abrasion or corrosion is excessive.

CHAPTER 7

OTHER BREAKWATERS

7-1. General. Protection for most coastal projects will probably be most advantageously provided by a structure of the rubble-mound, vertical wall or floating type; however, some projects may be best served by other unique structure types. It is beyond the scope of this manual to provide design guidance for all types of breakwaters. The pneumatic, hydraulic, and sloping float breakwaters have been chosen for inclusion herein, since they have generated more interest than most other lesser known structure types.

7-2. Pneumatic Breakwater System. The pneumatic breakwater concept was patented in 1907 (item 10). Wave attenuation is achieved by releasing compressed air through a submerged perforated pipe. Several prototype installations of this system have been described as successful. A few model studies were conducted prior to 1950, but the results were incomplete and in some cases contradictory.

a. Theoretical Analysis.

(1) Taylor conducted an analysis of the pneumatic breakwater, and his development became one of the most significant advances in this area of research (item 129). The investigation was formulated around the superposition of a uniform current of velocity, U , and thickness, h , on the velocity potential of a deepwater wave. It was assumed that air bubbles had little effect on the attenuation, and that the vertical current induced by the rising bubbles diffusing both upstream and downstream at the surface was solely responsible for the attenuation of the incident waves. Taylor's analysis was aimed at determining the current velocity necessary to attenuate waves of a given length, and he found that, for a given current, it was kinematically impossible to transmit waves shorter than a given length.

(2) Taylor modified the theory by using a triangular velocity distribution, which is more in accord with actual prototype distributions (item 130). To relate the velocity and thickness of the current to the air discharge and the submergence of the perforated pipe, the analogous solution for the convective currents above a horizontal line source of heat was used. The maximum velocity of the current U was found to be related to the air discharge q as

$$q = 0.00454U^3 \tag{7-1}$$

b. Small-Scale Experimental Studies. Several small-scale experimental studies conducted on pneumatic breakwaters (items 16, 36, 124, and 143) determined that the power required for discharging air through the pneumatic breakwater could be conveniently expressed by the dimensionless parameter

$$\Phi = \frac{(hp/ft)}{(\rho g^{3/2} L^{5/2})} \tag{7-2}$$

The horsepower per foot (hp/ft) at the orifices was computed from the expression

$$hp/ft = \frac{(q\gamma_w d_1)}{550} \tag{7-3}$$

where

ρ = density of water

g = acceleration of gravity

L = wavelength

q = unit air discharge at orifice

γ_w = unit weight of water

d_1 = submergences of orifices

(1) Effect of wave steepness. Wave steepness in the laboratory experiments varied from 0.02 to 0.08. It was found that the air requirement for a given attenuation was essentially independent of the wave steepness.

(2) Effect of orifice area. Straub, Bowers, and Tarapore investigated this effect with orifices of 1/8-, 3/16-, and 1/4-inch diameter (item 124). Test results indicated no pronounced change in the air requirements for the different orifice sizes.

(3) Use of multiple manifolds. Straub, Bowers, and Tarapore hypothesized that multiple parallel manifolds would be advantageous for attenuation of longer waves. This would provide a deeper surface current, thus enabling the breakwater to intercept the orbital motion over a greater part of the wavelength. Up to four manifolds were tested, but there appeared to be no advantage to using multiple manifolds. Actually, for lower discharges the airflow was not uniform and resulted in poor efficiency.

(4) Power requirement. For illustrative purposes, the horsepower required for a potential prototype installation was computed based on results of small-scale laboratory experiments of (item 124). Assuming an

installation depth of 40 feet and for various periods (wavelengths), attenuation as a function of applied horsepower per foot of breakwater is shown in figure 7-1. From this direct extrapolation of small-scale experimental data to prototype scale, it appears that the horsepower requirement would make operation very costly.

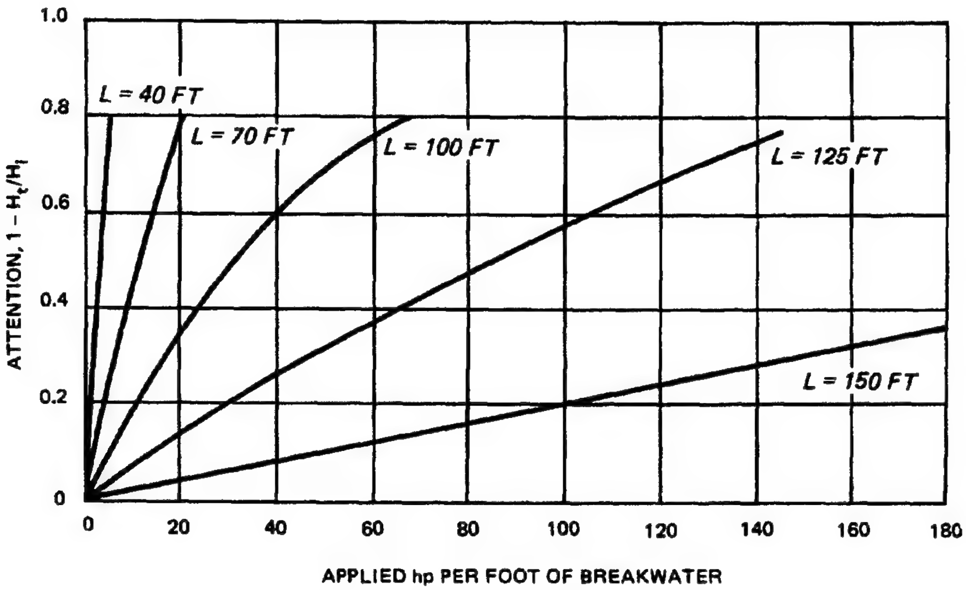


Figure 7-1. Effect of applied horsepower and wavelength L on effectiveness of the pneumatic breakwater at a 40-foot depth

c. Large-Scale Experimental Studies. Pneumatic wave attenuation systems have one distinct advantage in that they allow unrestricted passage over the breakwater. Sherk considered that the concept merited large-scale experimental investigation despite the large horsepower requirements predicted from the previous small-scale tests (item 119). Sherk's experimental study was conducted in 16 feet of water using various wave heights and periods. Wave periods ranged from 2.61 to 16.01 seconds, sufficiently covering the range of wave periods most often found in the open ocean. The larger scale tests indicated that approximately 20 percent less horsepower than was predicted from previous small-scale tests is needed to produce a like attenuation. Operation would still be costly, even with this small reduction in the power requirement.

7-3. Hydraulic Breakwater System. Hydraulic breakwaters achieve wave attenuation by discharging water under pressure through a manifold in a direction opposed to a train of surface gravity waves. The water jets diffuse, a horizontal current is formed, and a high degree of turbulence and mixing

occurs. Waves propagating into the current dissipate a portion of their energy by partial or complete breaking. Thus, the hydraulic breakwater is conceptually similar to the pneumatic breakwater except for the manner in which the horizontal current is formed. Performance of the hydraulic breakwater has been investigated for intermediate depth waves ($0.05 < d/L < 0.5$) (items 60, 125, and 146). The primary objectives of the two-dimensional hydraulic breakwater studies, described in items 60 and 123, were to obtain information concerning the effects of various parameters on wave attenuation, discharge, and horsepower requirements. Experimental data indicated that power requirements primarily depend on wavelength, water depth, and wave steepness, and submergence, spacing, and size of nozzles.

a. Effect of Relative Wavelength. Data were obtained for d/L values ranging from 0.2 to 1.0. For a constant level of attenuation, power requirements remained fairly constant as d/L decreased from 1.0 to 0.5, and then increased rapidly for smaller values of d/L, with the power requirement at $d/L = 0.2$, being seven times greater than that observed for $d/L = 0.5$.

b. Effect of Wave Steepness. Wave steepness was found to have an important effect on power requirements. For a constant level of attenuation and constant d/L, the required horsepower increased by a factor of about three as the incident wave steepness (H_i/L_i) increased from 0.02 to 0.08.

c. Effect of Jet Area. Jet nozzle cross-sectional area per linear foot of breakwater has been found to influence both the discharge and power requirements. Generally, power requirements decrease and the required discharge increases as the jet area is increased.

d. Efficiency. Herbich, Ziegler, and Bowers found that more power was required to attenuate relatively steep waves than flatter waves; however, the efficiency of the system, e, was found to be higher for the steeper waves (item 60). The efficiency e can be defined as

$$e = \frac{(P_i - P_t)}{P_j} \tag{7-4}$$

where

P_i = the power of the incident wave train

P_t = the power of the transmitted wave train

P_j = the power of the hydraulic jets

As illustrated in figure 7-2, efficiency varied with incident wave steepness H_i/L_i, relative wavelength d/L, and attenuation. Assuming an installation

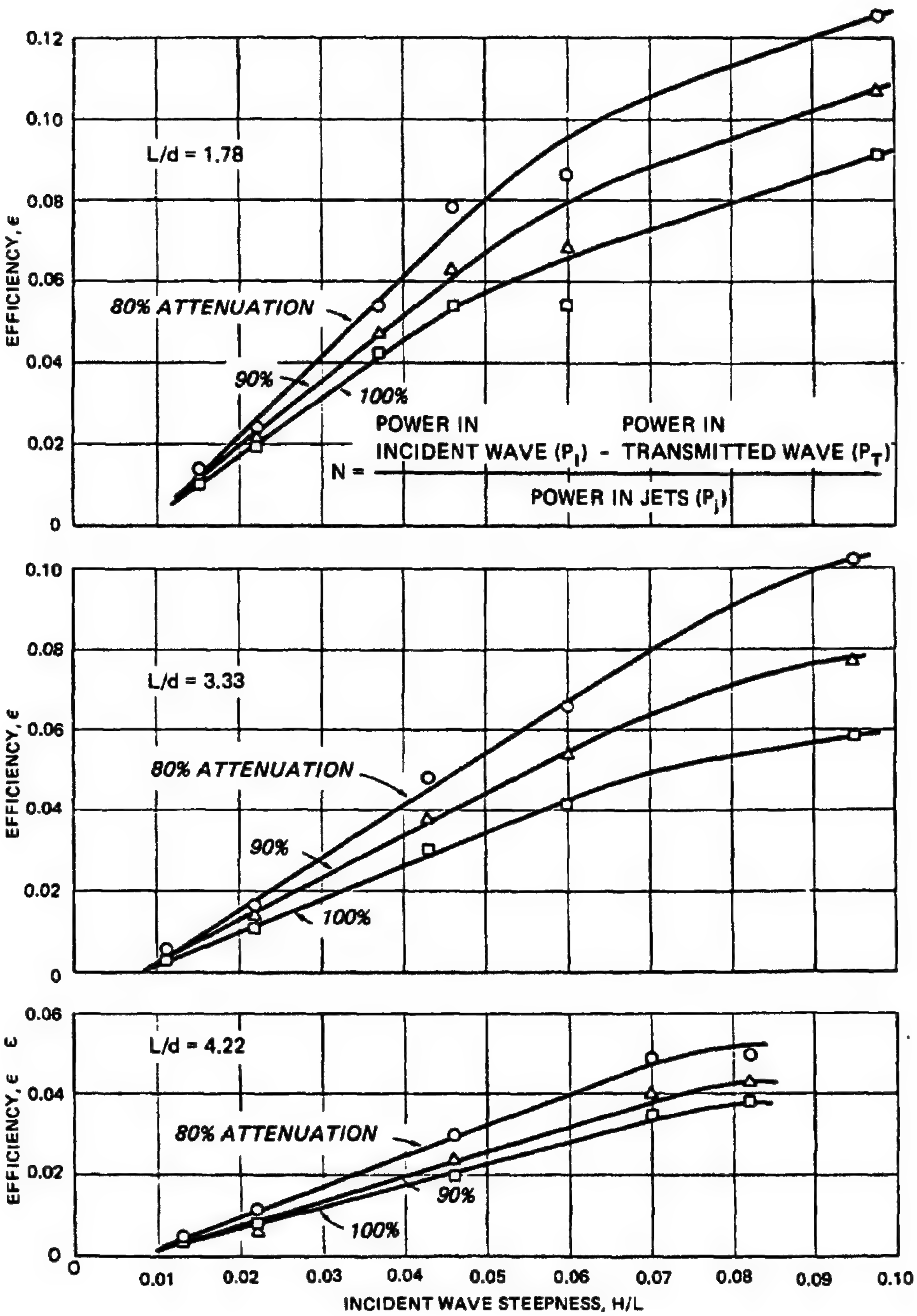


Figure 7-2. Efficiency of the hydraulic breakwater as a function of wave steepness, H/L , and relative wavelength, L/d

depth of 40 feet and for various wave conditions, attenuation as a function of applied horsepower per foot of breakwater is shown in figure 7-3. Similar to the pneumatic breakwater, the hydraulic breakwater's horsepower requirement would make operation very costly. It should be noted that neither the pneumatic nor the hydraulic breakwater have proven cost effective in a prototype installation.

7-4. Sloping Float Breakwater.

a. General. The sloping float breakwater (SFB) is a wave barrier that consists of a row of flat slabs or panels, with weight distribution such that each panel rests with one end above the water surface and the other end on the bottom. Hollow steel barges of the Ammi pontoon or Navy Lightered pontoon type afford one means of construction; however, various other types of construction are possible. Deployment of the pontoon-type structures would consist of assembling unballasted modules at the surface and then partially flooding the barges so that the stern sinks and rests on the bottom and the bow floats above the water surface. The height of protrusion of the bow above the water surface (freeboard) is controlled by flooding a selected number of pontoons. Barges are sited so that the bow faces into the primary direction of wave attack, and mooring lines are attached between it and a bottom anchor. Figure 7-4 is a conceptual sketch of the SFB. Performance of the SFB has been investigated in hydraulic model tests using monochromatic waves (items 107, 109, and 110). Hydraulic model tests of the concept using spectral waves are described in item 30.

b. Wave Attenuation Capabilities.

(1) In hydraulic model tests (item 30) an investigation was conducted of a wide range of wave periods, wave heights, and water depths. Tests were conducted with a 1V:50H bottom slope using shallow-water wave spectra characteristic of the North Carolina coast. Even though tests were site specific, it is felt that they should provide good general guidance to expected SFB performance due to the wide range of conditions investigated and the commonality of shallow-water wave spectra for similar wave heights and periods. Therefore, findings discussed within item 30 are summarized in the following paragraphs.

(2) The SFB's selected for testing were Navy Lightered pontoon-type barges 89.6 and 118.4 feet long, weighing 134,000 and 177,000 pounds, respectively. Both were 21 feet wide and 5 feet deep. Tests were conducted with about 5 feet of freeboard. This condition required 366,000 and 467,000 pounds of seawater ballast for the 89.6- and 118.4-foot barges, respectively.

(3) Important geometric and dynamic details of the prototype barges were considered in the design and construction of the model section. Overall prototype dimensions were exactly reproduced, and all major parameters that control rigid body dynamic response such as weight, center of gravity, mass moments of inertia, and angle of inclination were reproduced within

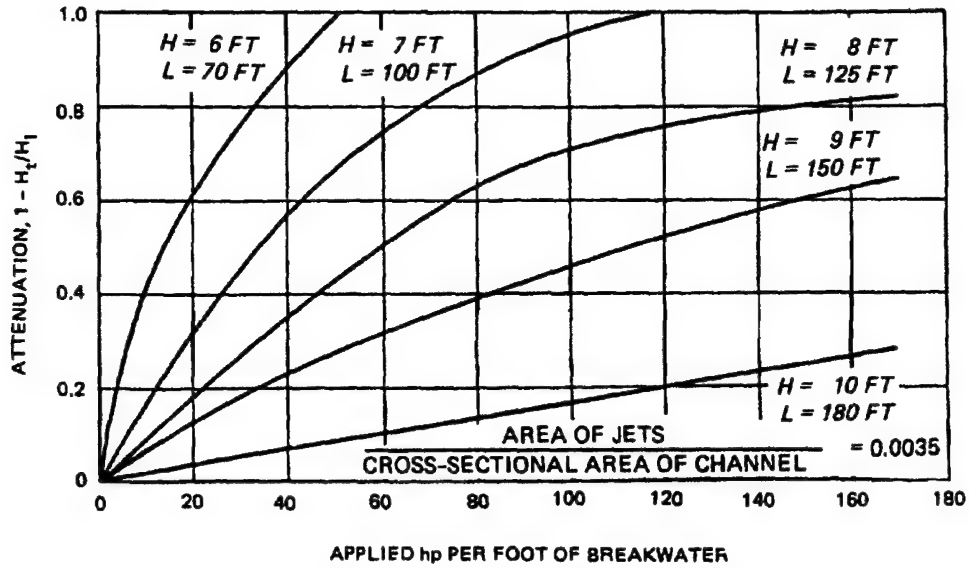


Figure 7-3. Effect of applied horsepower and wavelength, L , on effectiveness of the hydraulic breakwater at a 40-foot depth (H_t = transmitted wave height)

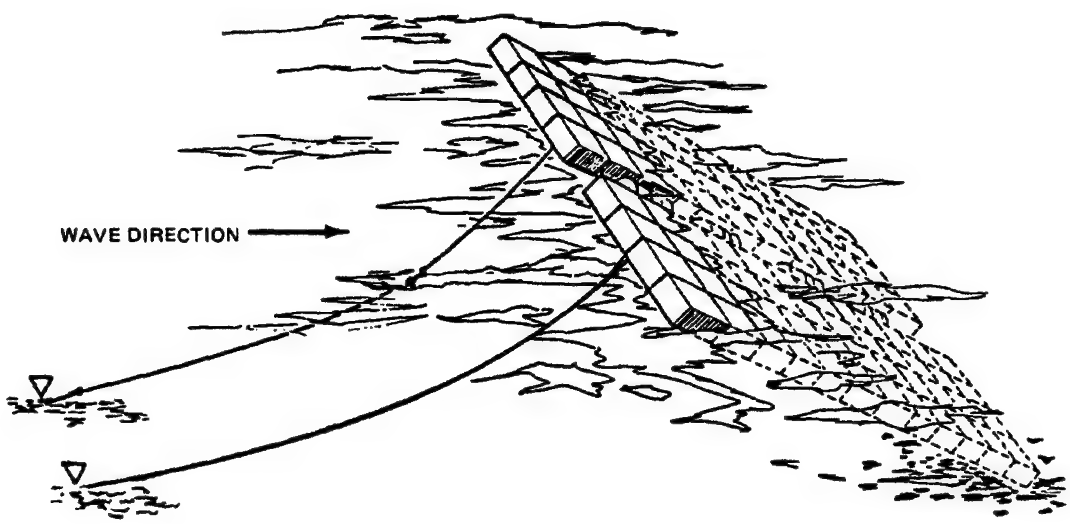


Figure 7-4. An artist's conception of the sloping float breakwater (SFB)

± 1.0 percent. Barges were moored to a bottom anchor using a 150-foot-long braided nylon line with a breaking strength of 230,000 pounds. Nonlinear restoring force characteristics of the mooring line were simulated in the model with a series of springs.

(4) All tests were conducted with spectral waves. Peak periods (T_p) of the spectra ranged from 6 to 14 seconds, and the significant wave heights (H_s) were 2, 4, 6, and 8 feet. The structures were anchored in water depths of 13, 15, 18, and 21 feet.

(5) Examination of wave test results from item 30 shows that coefficients of transmission (C_t) and peak mooring force (F_p) appear to primarily depend on wave period or length, SFB length, and water depth, i.e.,

$$C_t = f(T_p, L_{SFB}, d)$$

$$F_p = f(T_p, L_{SFB}, d)$$

The variables T_p, L_{SFB}, and d are defined as the peak period of the spectra, length of the sloping float breakwaters (SFB) and water depth, respectively. Figures 7-5 and 7-6 present C_t and F_p, respectively, as a function of wave period for SFB lengths of 118.4 and 89.6 feet and a water depth of 18 feet. Figures 7-7 and 7-8 present C_t and F_p, respectively, as a function of water depth, for SFB lengths of 118.4 and 89.6 feet and wave periods ranging from 6 to 14 seconds.

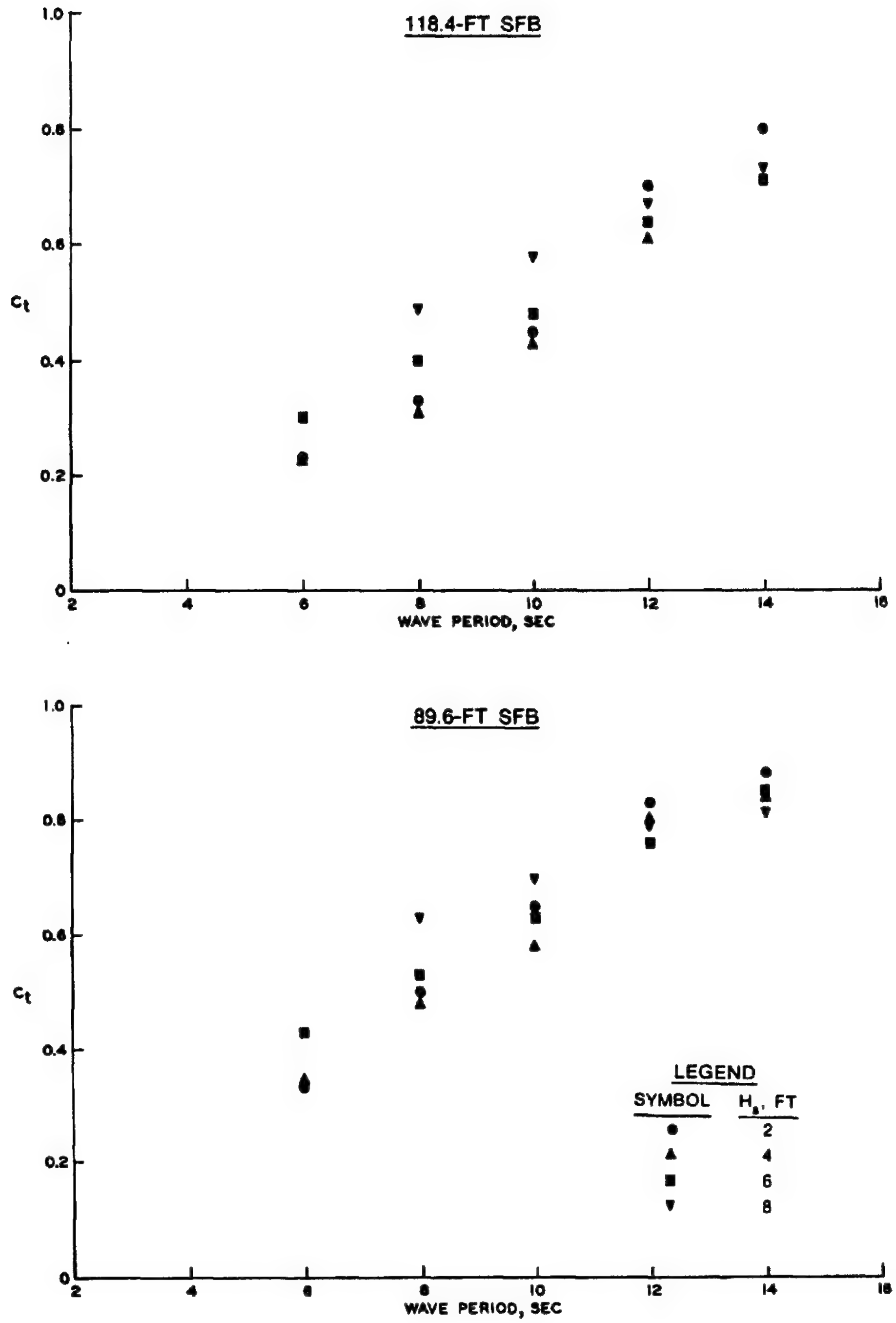


Figure 7-5. Wave attenuating capabilities of the SFB in an 18-foot water depth

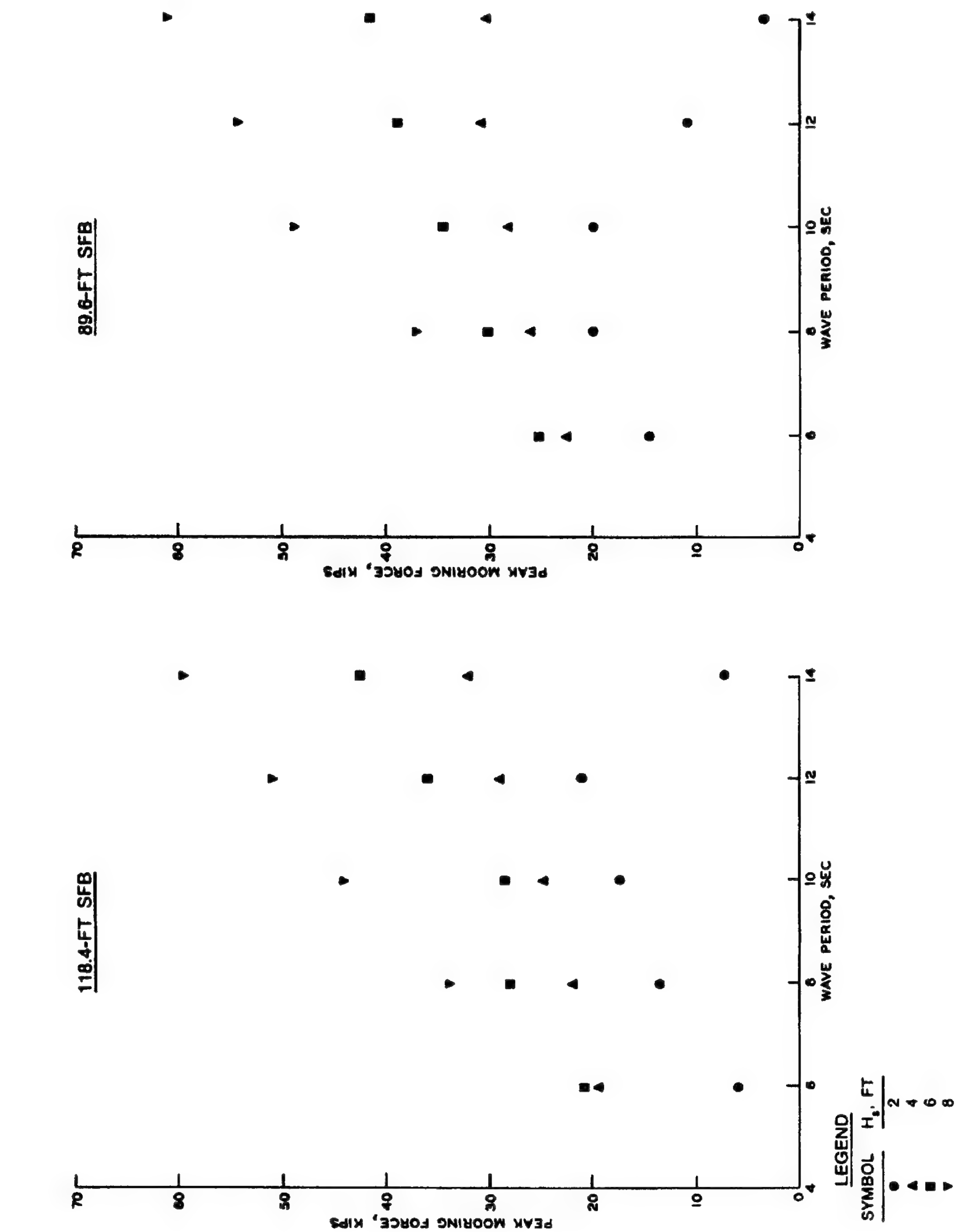


Figure 7-6. Peak mooring forces developed by the SFB in an 18-foot water depth

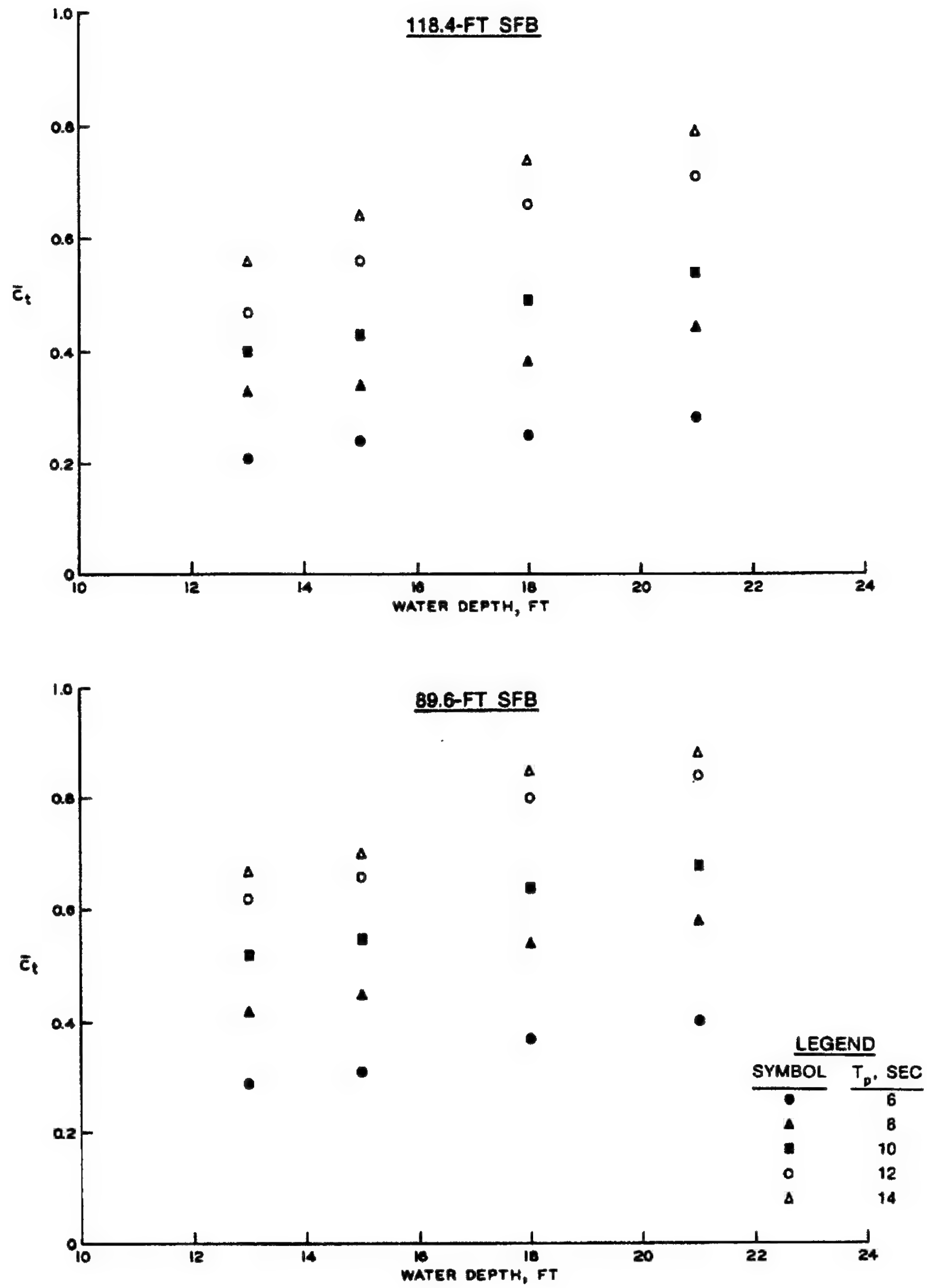


Figure 7-7. Wave attenuating capabilities of the SFB as a function of water depth, wave period, and breakwater length

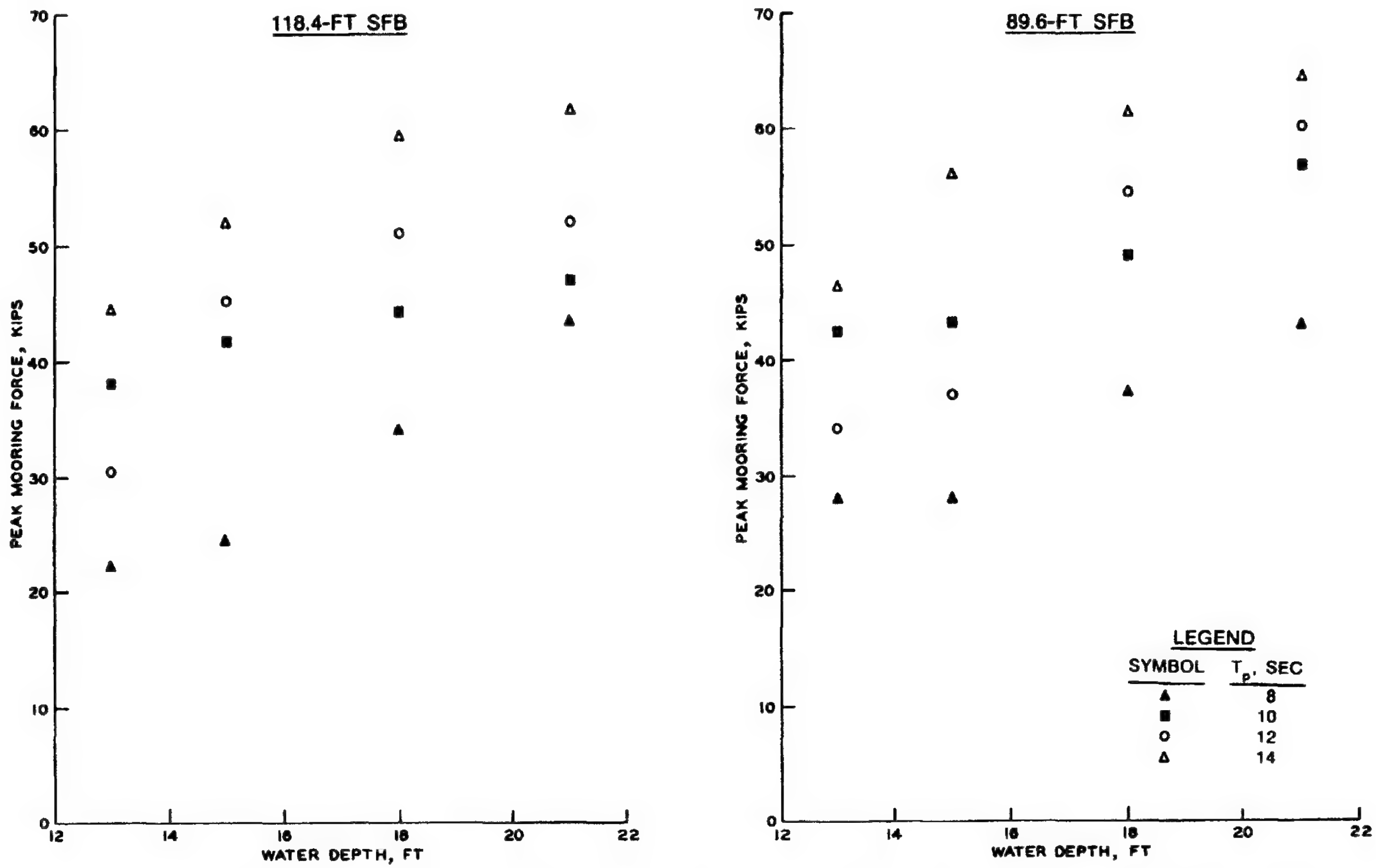


Figure 7-8. Peak mooring forces developed by the SFB under attack of 8-foot waves as a function of water depth, wave period, and breakwater length

CHAPTER 8

ENVIRONMENTAL IMPACTS

8-1. General. A common characteristic of breakwaters and jetties is their location in dynamic, high energy environments. Physical features of the environment where breakwaters and jetties are typically constructed reflect hydrodynamic and sedimentological conditions that have attained a dynamic equilibrium, a state of continuous change which remains balanced around some average set of conditions. Environmental impacts will occur as the system is initially imbalanced by the presence of the structure(s), and then returns to a new set of dynamic equilibrium conditions. Potential environmental impacts associated with these structures can be sorted into the following categories, all of which are interrelated to some degree: physical impacts, water quality impacts, biological impacts, and socioeconomic and cultural impacts (items 20, 21, and 97). The magnitude of severity of each type of impact can be expected to vary over short or long spans of time. Each category of impact is briefly discussed below. Because breakwaters and jetties generate essentially similar impacts, they are treated jointly.

8-2. Physical Impacts.

a. Breakwater or jetty construction is invariably accompanied by localized changes in the hydrodynamic regime. In the case of tidal inlets with either single or double jetty systems, for example, longshore currents are deflected beyond the seaward end of the structure(s) and, depending on the orientation of the structure(s) to the inlet, water circulation through the inlet is altered. The presence of a structure adjacent to a channel may cause an increase or decrease in the minimum channel cross-sectional area, which in turn is related to water current velocities and availability of sediments. Changes in hydrodynamic regime such as these provide the driving force for additional physical, water quality, and biological impacts. Breakwater configuration often produces a semiconfined water basin in which water current flows are reduced, thereby affecting the area's flushing rate. This is an important design consideration when contaminants might be present, as is often the case in small boat harbors or larger docking facilities. Breakwaters and jetties may alter water circulation patterns in a manner such that areas conducive to sediment erosion and/or deposition are created or redistributed. The rates of shoreline erosion and accretion are proportional to the magnitude of the littoral sediment transport process peculiar to a given site. Spatial extent of resultant shoreline alteration is a function of the structure's effectiveness as a barrier to littoral sediment drift as determined by the structure's orientation to the shoreline. Formation, degradation, or translocation of bars, shoals, or ebb tidal deltas are also direct results of altered hydrodynamic regimes (items 80 and 138). Another potential physical impact involves migration of channel thalwegs, particularly following construction of single jetties at tidal inlets. Predictions of changes in hydrodynamic regime can be obtained by means of physical or numerical hydrodynamic modeling investigations supplemented by experience with historical or existing field situations.

b. Physical impacts can be summarized as:

(1) Stabilized hydrodynamic regime.

(2) Stabilized bottom topography and shoreline configuration.

(3) Stabilized minimum channel cross-sectional area.

(4) Stabilized channel thalweg position.

8-3. Water Quality Impacts.

a. During the construction of a breakwater or jetty, suspended sediment concentrations may be elevated in water immediately adjacent to the operations. In many instances, however, construction will be occurring in naturally turbid estuarine or coastal waters. Plants and animals residing in these environments are generally adapted to, and very tolerant of, high suspended sediment concentrations. The current state of knowledge concerning suspended sediment effects indicates that anticipated levels generated by breakwater or jetty construction do not pose a significant environmental impact. Limited spatial extent and temporal duration of turbidity fields associated with these construction operations reinforce this statement. However, when construction is to occur in a clearwater environment, such as in the vicinity of coral reefs or seagrass beds, precautions should be taken to minimize the amounts of resuspended sediments. Organisms in these environments are generally less tolerant to increased siltation rates, reduced levels of available light, and other effects of elevated suspended sediment concentrations.

b. Indirect impacts on water quality may result from changes in the hydrodynamic regime. In addition to consideration of contaminant problems caused by reduced flushing rates, fluctuations in parameters such as salinity, temperature, dissolved oxygen, and dissolved organics may be induced by construction or by the actual presence of a structure. Potential water quality impacts should be evaluated with reference to the ecological requirements of important biological resources in the project area.

c. Potential water quality impacts can be summarized as:

(1) Temporary elevated suspended sediment concentrations.

(2) Altered levels of salinity, temperature, dissolved oxygen, etc.

8-4. Biological Impacts.

a. Biological impacts are inherently difficult to quantify. Impacts, indicated by changes in occurrences and abundances of organisms, may be masked by background "noise" due to seasonal variations in populations, ecological succession events, and natural perturbations (e.g. storms, harsh winters,

etc.). The types of biological impacts discussed below range in their order of presentation from well-established to highly speculative. Impacts discussed in paragraphs b and c deserve consideration in connection with almost all breakwater and jetty construction projects, whereas those that follow merit consideration only when sufficient cause for concern has been demonstrated for a given project.

b. Measurable amounts of bottom habitat are physically eradicated in the path of breakwater or jetty construction. Given an example toe-to-toe width of 125 feet, one linear mile of typical rubble structure replaces approximately 15.2 acres of pre-existing bottom habitat. This loss of benthic (bottom) habitat and associated benthos (bottom dwelling organisms) is more than offset by the new habitat represented by the structure itself and by the reef-like community which becomes established thereon. Submerged portions of breakwaters and jetties, including intertidal segments of coastal structures, function as artificial reef habitats and are rapidly colonized by opportunistic aquatic organisms (items 139 and 144). Over the course of time, structures in marine, estuarine, and most freshwater environments develop diverse, productive biological communities. A majority of large breakwaters and jetties are rubble-mound structures, which represent excellent spawning, nursery, shelter and/or foraging habitat for numerous desirable fish and shellfish species (item 68). This development of a reef-like community can definitely be viewed as a beneficial project impact, the scale of which will vary among regional locations.

c. Water currents and turbulence along the base of the structure can produce a scouring action which prevents utilization of that habitat area by most benthic organisms. This effect is largely confined to the bottom immediately adjacent to the structure and may occur along only a portion of the perimeter, such as along the channel side of an inlet's downdrift jetty (item 81).

d. One speculative source of biological concern related to altered hydrodynamic regimes at jettied coastal inlets involves transport of egg and larval stages of fish and shellfish. Eggs and larvae of many important sport and commercial species are almost entirely dependent upon water currents for transportation from offshore spawning areas through coastal inlets to estuarine nursery areas. Jetties displace the entrance to an inlet forming a potential barrier to eggs and larvae, particularly those carried by longshore currents. Eddies or lee areas created in the vicinity of jetties may act as sinks in which nonmotile stages become trapped or are delayed. Results of hydraulic modeling studies have been inconclusive, and field studies addressing the problem are nonexistent. Several studies have documented successful movement of organisms through jettied inlets (item 38), but pre- versus post-construction data are unavailable upon which to base comparisons. Historically, in view of the fact that numerous structures have been in place for quite a long period with no apparent decline in estuarine dependent species attributable to their presence, a case can be made that such impacts, even if real, are insignificant. Similar concerns have been voiced with

regard to the movements of juvenile and adult stages of various fish and shellfish. Because these are generally highly motile forms the probability of negative impact is even less significant.

e. Coastal rubble structures provide substratum for the establishment of artificial reef communities. As such, breakwaters and jetties serve as a focal point for aggregations of fish and shellfish which graze on sources of food or find shelter there. Many species are attracted to the structures in numbers, as evidenced by the popularity of breakwaters and jetties as sport fishing locations.

f. Potential biological impacts can be summarized as follows:

(1) Loss of benthic habitat and benthos in the area covered by the structure(s).

(2) Displacement of benthos due to scouring effects.

(3) Development of plant and animal communities on the substratum provided by the structure(s).

(4) Altered transport of egg and larval stages of fish and shellfish through coastal inlets.

(5) Altered movement patterns of juvenile and adult stages of fish and shellfish.

8-5. Short- and Long-Term Impacts.

a. Actual construction activities for breakwaters and jetties entail several months to several years of effort. During this period, a number of impacts of durations generally less than several days or weeks may occur. These impacts will vary in type and frequency from project to project. For example, temporary or permanent access roads may have to be built to allow transportation of heavy equipment and construction materials to the site. Grading, excavating, backfilling, and dredging operations will generate short-term episodes of noise and air pollution, and may locally disturb wildlife such as nesting or feeding shorebirds. Project planning should, to the extent practicable, schedule events to minimize disturbances to waterfowl, spawning fish and shellfish, nesting sea turtles, and other biological resources at the project site. Precautions should also be exercised to reduce the possibility of accidental spills or leakages of chemicals, fuels, or toxic substances during construction operations. Effort should be expended to minimize the production and release of high concentrations of suspended sediments, especially where and when sensitive biological resources such as corals or seagrasses could be impacted. Dredging of channels in conjunction with breakwater or jetty projects presents a need for additional consideration of short-term impacts as related to resuspended sediment effects.

b. Long-term impacts of breakwater or jetty construction are less definitive or predictable. Ultimate near field effects on littoral sediment transport can be expected to become evident within several seasonal cycles. These effects will vary according to the specific environmental setting and engineering design. For example, sediments accumulated in a deposition basin adjacent to a jetty weir can be used periodically to renourish adjacent erosional beaches. Consequences of construction on far field downdrift sediment processes are presently speculative. Also, because rubble-mound structures tend to become less permeable as they age, long-term shifts in distribution of benthic habitats at a project site may occur.

8-6. Socioeconomic and Cultural Impacts. A basic incentive for constructing breakwaters or jetties is to improve safety conditions for waterborne traffic through inlets and passes. This is the primary beneficial impact associated with construction. Other potential socioeconomic or cultural impacts are the presence of both archeological artifacts and cultural assets at a given project site. Where identified, these properties are given appropriate protection against possible loss or disturbance. Aesthetic quality of the structural design for the project also receives consideration. This is largely determined by subjective criteria, and provides a measure of how well the structure blends with its natural setting. Few options exist to minimize the visual contrast structures present against the backdrop of the coastal environment. Visual impacts, however, can be somewhat offset by improved access to the shoreline for fishing, swimming, diving, sightseeing, and other recreational activities. Attraction of many game fish to breakwaters and jetties underscores the value of these structures as desirable fishing spots, particularly for the nonboating public. High public utilization patterns of breakwaters and jetties also serve to support bait and tackle shops and to further stimulate local economies.

8-7. Evaluation of Project Alternatives. Each breakwater or jetty project scenario should incorporate engineering design, economic cost-benefit, and environmental impact evaluations from the inception of planning stages. All three elements are interrelated to such a degree that efficient project planning demands their integration. Environmental considerations should not be an afterthought. Structure design criteria should seek to minimize negative environmental impacts and optimize yield of suitable habitat for biological resources. This can be achieved by critical comparisons of a range of project alternatives, including the alternative of no construction at all. Various engineering design features can be incorporated into an optimal ecological alternative. For example, selection of a design specification for a less steep alternative of side-slope angle will maximize the availability of intertidal and subtidal habitat surface area. The size class of stone used in armor layers of rubble structures is another engineering design feature that has habitat value consequences. The large armor material results in a heterogeneous array of interstital spaces on the finished structure.

CHAPTER 9

OTHER CONSIDERATIONS

9-1. General. Unique characteristics of individual projects may necessitate additional considerations other than those presented in this and other chapters. These circumstances will draw heavily on the creativity, engineering judgment, and experience of both designer and reviewer. Considerations presented in this chapter are aesthetics, fishing platforms, aids to navigation, and construction methods. All of these will have application to most breakwater and jetty designs.

9-2. Aesthetics. Breakwaters and jetties should be pleasing in form as well as functional. Good workmanship and close adherence to design grades contribute to the aesthetics of these structures. Repair sections should be geometrically similar to the original structure. Public reaction to existing projects can serve as input to the design. Examples of projects which require aesthetic consideration are scrap-tire breakwaters, which may be viewed as unsightly, or high-crested structures, which may block a scenic ocean view.

9-3. Fishing Platforms. Breakwaters and jetties normally provide an excellent habitat for fish, thus recreational fishermen are attracted to the structures. It may be very difficult to provide a safe fishing area, especially on some types of structures such as low-crested, rubble-mound breakwaters or jetties. Single-pontoon floating breakwaters provide an excellent fishing platform. Where safe and justified, designs for breakwaters and jetties should include accommodations for recreational fishing.

9-4. Aids to Navigation. Prior to construction of any breakwater or jetty which may necessitate new aids to navigation or affect existing aids, complete information on the proposed structure will be furnished directly to the Coast Guard district commander. This information shall include (a) information in regard to the authorization of the construction of a breakwater or jetty, including a copy of the project document, and (b) the proposed construction schedule; maps showing the final location of the structure should be furnished when the work is definitely undertaken.

9-5. Construction Methods. Typical methods of constructing rubble-mound breakwaters and jetties include (a) placement of materials with a crane operated from the structure's crest and the materials either barged to the site or transported along the crest from land; (b) construction of a temporary trestle above the structure from which a crane places materials that have been transported along the trestle from land or barged to the site; and (c) construction from floating plant, i.e., transportation of materials to the site by barge and placement with a barge-mounted crane. Some of the smaller materials, such as the bedding and core, may be dumped directly on the structure. Concrete armor units are always individually placed, with care taken to assure the units are not overstressed and uniform coverage of the structure is

achieved. Generally, construction methods should be chosen to give the needed degree of quality control at a minimum cost.

CHAPTER 10

DESIGN OPTIMIZATION

10-1. Design Optimization.

a. The project design life and design level of protection are required before the design conditions can be selected. The economic design life of most breakwaters and jetties is 50 years. Level of protection during the 50-year period is usually selected by an optimization process of frequency of damages when wave heights exceed the design wave and the cost of protection. The elements that are to be considered in an economic optimization or life cycle analysis are as follows:

(1) Project economic life.

(2) Construction cost for various design levels.

(3) Maintenance cost for various design levels.

(4) Replacement cost for various design levels.

(5) Benefits for various design levels.

(6) Probability for exceedance for various design levels.

b. The design level for a breakwater or jetty is usually related to wave characteristics and water levels. The severity of these events has a statistical distribution that can be ordered into a probability of exceedance. The exceedance probability is plotted against the design level (figure 10-1).

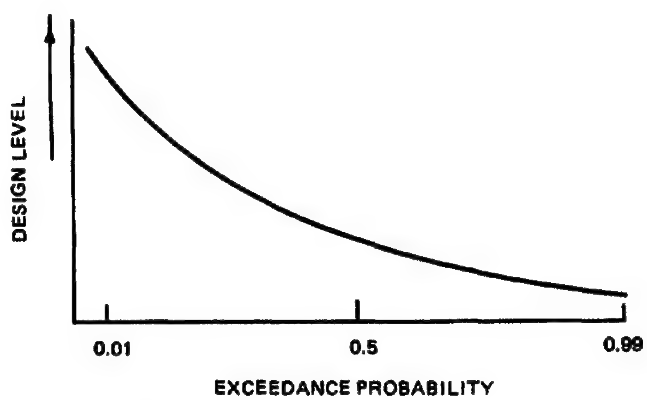


Figure 10-1. Exceedance probability versus design level

c. A series of project designs and cost estimates are developed for various design levels (water levels and wave heights). Construction costs are then converted to annual cost. Maintenance costs can be estimated by using table 4-4 and expected wave height exceedance frequencies illustrated in paragraph 4-17. This maintenance cost should be compared with maintenance of similar existing projects to assure realistic values.

d. Some designs may call for partial or total replacement of a project feature one or more times during the project economic life. Average annual replacement costs are obtained by estimating the replacement years, determining replacement cost and converting to present worth. The present worth value of the replacement is then converted to average annual cost by using appropriate interest rates and economic project life. The project cost curves usually look like those in figure 10-2.

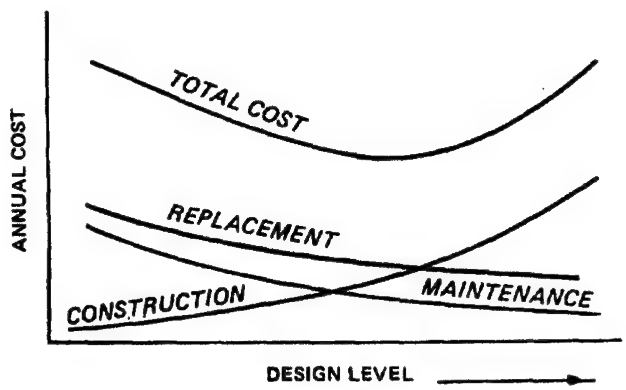


Figure 10-2. Project cost curves

e. Benefits are compared with cost to determine the optimum economic design. Figure 10-3 shows this benefit/cost comparison. Normally, the design level associated with the maximum net benefits will be selected for project design.

10-2. <u>Alternative Structures</u>.

a. The design process should include consideration of all alternative types of breakwaters which are suitable for the site conditions. These suitable alternatives can be:

(1) Various types of structures, such as floating or rubble-mound breakwaters.

(2) Alternative types of armor units for rubble-mound breakwaters.

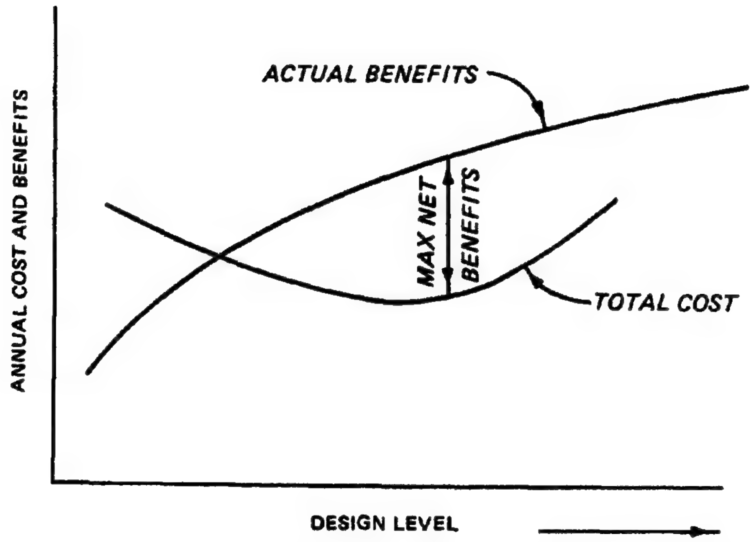


Figure 10-3. Benefits and cost versus design level

(3) "Overdesigning" rubble-mound armor units.

"Overdesigning" can greatly increase the factor of safety and reduce maintenance cost at no increase in cost. An example of this overdesign analysis is presented in item 141, where a comparison is made of dolos units which were designed for K_D = 25 (i.e., stable for design wave) and a second group designed for K_D = 13.6 (i.e., overdesigned). The following variables were used in this analysis:

Dolos stability coefficient = K_D = 25 and 13.6.

Structure slope = Cot α = 1.5, 2.0, 2.5, and 3.0.

Concrete unit weight = 150, 160, and 170 pounds per cubic foot.

b. Figure 10-4 shows the analysis for these variables based on rehabilitation cost for Humboldt jetty at Eureka, California, in 1970-72. The figure presents total first cost for 100 feet of structure as a function of dolos weight, structure slope, and concrete unit weight. Each point in the figure represents a solution to the design problem. One solution (Example 1 in figure 10-4), using the curves for K_D = 13.6 , would be to construct the jetty with a slope of 1 on 2 of concrete with a unit weight of 160 pounds per cubic foot which requires a 5.2-ton dolos for armor against the 18-foot design wave. The cost for 100 feet of structure armored with a 5.2-ton dolosse is estimated at about $618,000. Another solution to the design problem (Example 2 in figure 10-4) would be to use a 7-ton dolos having a unit weight of 155 pounds per cubic foot placed on a 1-on-1.75 slope. The estimated cost of this solution per 100 feet of structure is $565,000.

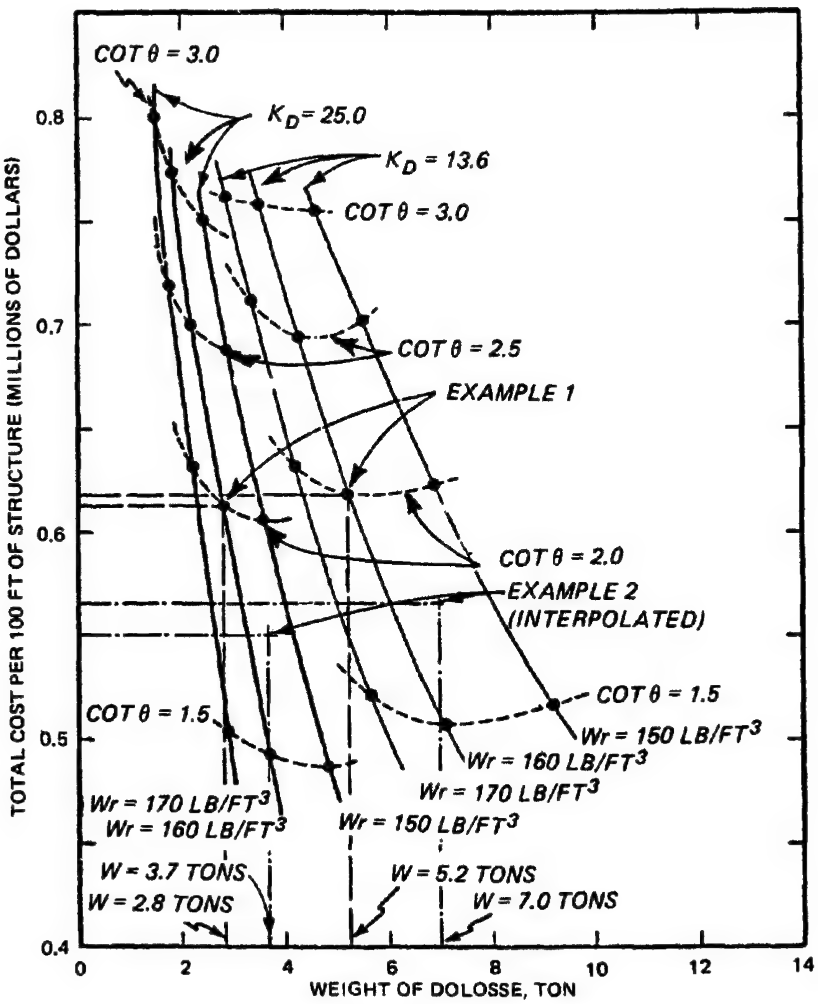


Figure 10-4. Total cost of 100 feet of structure as a function of structure slope, concrete unit weight, and dolosse weight for K_D = 13.6 and K_D = 25.0

c. When the stability coefficient is increased to K_D = 25.0 , the family of curves to the left in figure 10-4 represents solutions to the design problem. The required dolos weight has been nearly halved for equivalent conditions of structure slope and concrete unit weight. The cost per 100 feet of structure, however, has not changed appreciably; e.g., using K_D = 25.0 for conditions cited in Example 1 below with a structure slope of 1 on 2 and a concrete unit weight of 160 pounds per cubic foot, the required dolos weight has been reduced from 5.2 to 2.8 tons but the estimated cost has only decreased from $618,000 to $612,000 per 100 feet of structure. In Example 2, the required dolos is now only 3.7 tons rather than 7 tons but the estimated cost has only decreased from $565,000 to $550,000 (2.7 percent) per 100 feet.

In fact, for some conditions of structure slope and concrete unit weight the cost actually increases for the larger stability coefficient and smaller armor units. This generally occurs for flatter slopes and higher values of concrete unit weight.

d. The explanations for the relatively small change in cost with smaller armor units are that (1) the cost of the armor layer may represent a relatively small percentage of the total cost of the structure, especially for flat-sloped structures that have large quantities of core material, and (2) the relative cost of labor compared with the cost of materials used to construct armor units is high and results in an increase in the cost of armor. Labor costs in casting concrete armor units are sensitive to the number of units that need to be formed, stripped from forms, reinforced (if necessary), transported, and placed on the structure. The cost of materials, on the other hand, is simply proportional to the amount of materials needed. As the size of armor units decreases, the number of units required to cover a given structure surface area increases, and, along with it, the cost of labor to form, strip, reinforce, transport, and place the units; conversely, the amount of concrete, reinforcing, etc., required to cover a given area in armor will decrease with decreasing armor unit size. Whether or not a cost saving is realized by decreasing armor unit size depends on whether the savings achieved by using less materials exceed any increase in labor costs resulting from using more armor units. The relative cost of labor versus materials is thus an important factor in establishing the optimum size armor unit. As the relative cost of labor increases, it becomes more economical to design using fewer, larger units; i.e., overdesigning the armor.

e. It is recommended that designers of rubble-mound structures work closely with cost estimators to ensure that an optimum level of design is achieved. This can only be obtained if a range of design wave heights and corresponding structure designs is evaluated.

CHAPTER 11

MODEL STUDIES

11-1. General. Hydraulic model investigations are an invaluable tool in the final design of breakwaters and jetties. Design guidance presented herein is sufficient for selection of structure type and preliminary design; however, proposed final designs may be optimized or at least check-tested in a hydraulic model study. The decision to conduct a model study should be based on an evaluation of such factors as complexity of bathymetry and structure geometry, estimated project costs, and consequences of failure. Experience has shown that site-specific model studies generally yield an excellent return on their original investment, either through savings in original construction costs as a result of optimization, or savings in repair and/or replacement costs as a result of identifying unsatisfactory designs prior to their construction.

11-2. Purpose of Model Tests. Hydraulic model tests of breakwater and jetties generally are conducted to

a. Determine minimum stable armor weights for rubble-mound structures.

b. Optimize the armor slopes and crown elevation of rubble-mound structures.

c. Quantify wave heights on the harbor-side of rubble-mound structures created by overtopping and transmission through the structure.

d. Determine wave transmission characteristics of floating breakwaters.

e. Measure mooring forces exerted by floating breakwaters.

11-3. Field Data Required.

a. In the design of hydraulic models, it is important that adequate information is available about the site so that major problems confronting the field design engineer are clearly understood by the laboratory engineer. The purpose and scope of model studies should be determined to the fullest extent possible at the outset. Model design and the testing program then can be better directed toward solution of those parts of the overall problem that are the most critical and are best suited for investigation by a hydraulic model. In addition to general information about the design problems (to determine the purpose and scope of the model investigation), the design, construction, and operation of models of coastal structures exposed to wave action require (1) detailed information on the geometry of the structure and materials of which the structure will be composed, (2) information concerning the bottom aterials upon which the structure will be situated, (3) the bottom contours along the alignment of the structure and seaward of the structure to a water

depth of nearly one-half the maximum wavelength, and (4) statistical data to determine the frequency of occurrence of waves with different heights and periods at the structure site.

b. The normal water depths at the structure site and the range of water surface elevations about the selected still-water level are important variables in the design of coastal structures, selection of design waves, and selection of model test conditions. Thus, statistical data of tidal ranges, wind setup, or storm surge are necessary for the design and efficient operation of models for all types of coastal structures.

11-4. Selection of Model Scale. During the planning and design phases of a hydraulic model study of breakwaters or jetties, the model scale must be determined. Scale selection normally is based on the following factors:

a. Preclusion of stability scale effects.

b. Size of model armor units available compared with the estimated size of prototype armor units required for stability.

c. Depth of water at the structure.

d. Capabilities of the available wave tank and wave generator.

Depending on the size of structure and wave conditions being represented, typical values of the model scale or length ratio (L_r) range from 1:25 to 1:50. Thus, models are typically from 25 to 50 times smaller than their prototype counterparts.

11-5. Model Laws. Following selection of the linear scale, the model is designed and operated in accordance with Froude's model law (item 121). Scale relations used for design and operation are given in the following tabulation:

Characteristic	Dimension[a]	Scale Relation[b]
Length	L	L_r
Area	L^2	$A_r = L_r^2$
Volume	L^3	$V_r = L_r^3$
Time	T	$T_r = L_r^{1/2}$
Force	F	$F_r = L_r^3$

[a] Dimensions are in terms of force (F), length (L), and time (T).

[b] The subscript r means "ratio."

11-6. Wave Generators. Model waves are normally generated by vertical-motion, plunger-type wave generators, horizontal-motion, piston-type wave generators; hinged-motion, flapper-type wave generators; or some combination of these. In each case, the movement of the wave board causes a displacement of water incident to its motion, which can be monochromatic or spectral.

11-7. Bottom Slope. Proposed breakwaters and jetties are normally fronted by variable bottom slopes. Effects of the bottom slope are important if the structure will be exposed to depth-limited breaking wave attack, since the height of depth-limited breaking waves increases as the slope becomes steeper. Therefore, the steepest slope fronting the structure is usually chosen for representation in the model.

11-8. Method of Constructing Test Sections. Model breakwater and jetty sections are constructed to reproduce as closely as possible results obtainable by a general coastal contractor. Core material, dampened as it is dumped by bucket or shovel into the flume, is compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype structure. Once the core material is in place, it is sprayed with a low-velocity water hose to ensure adequate compaction of the material. Underlayer stone is then added by shovel and smoothed to grade by hand or with trowels but it is not packed in place. Armor units used in the cover layer are placed by hand, usually in a random manner; i.e., laid down in such a way that no intentional interlocking of the units is obtained. Model elevations can be controlled with an engineer's level to a tolerance of $\pm$ 0.005 foot.

11-9. Still Water Levels. Still water levels (swl's) for breakwater and jetty models are selected so that the various wave-induced effects that are dependent on water depth are accurately reproduced. These effects include armor stability, amount of wave overtopping, and wave energy transmission through the structure. Generally, a range of swl's will be investigated.

11-10. Wave Characteristics. In planning the testing program for model investigation of wave-action problems, it is necessary to select wave dimensions that will allow a realistic test of the proposed structure. Wave transmission and overtopping tests are conducted for a range of wave conditions, thereby allowing determination of the structure's effectiveness as a function of wave height and period. Stability of the structure is investigated for the most severe wave conditions expected to occur during its design life.

CHAPTER 12

PERFORMANCE MONITORING

12-1. Need. Considering the investment costs, the essential functions breakwaters and jetties perform in protecting navigation and landward facilities, and the impacts such structures have on their surroundings, some type of performance or condition monitoring is often required. Projects containing new designs, which may have generic applications, should be monitored to evaluate the new design aspect. Projects designed with the aid of mathematical or hydraulic model studies, or utilizing new design theory, should be monitored to provide prototype verification of model studies or evolving technology. This will provide information, not only beneficial to determining the need for future maintenance or modification of the breakwater or jetty project itself, but will also aid in designing future similar structures. The Monitoring of Completed Coastal Projects (MCCP) program has been established for the above purposes. The MCCP program is managed by HQ, USACE through the Hydraulic Design Branch (ER 1110-2-8151).

12-2. Scope. Most breakwaters and jetties receive some monitoring. This may consist of only a periodic site visit by an engineer or may include accurate topographic and bathymetric surveys; land-based and aerial photographs of structure features; instrument recordings of wave characteristics, tides, currents, and other environmental factors; diver inspections; and/or side-scan sonar records of subaqeous features. The monitoring effort may also include the shores on both sides of the structure, and the offshore. Frequency and duration of monitoring efforts will depend on the purpose and objective of the monitoring.

12-3. Inspection. Periodic inspections will be made of breakwater and jetty structures to determine their condition, adequacy to serve their intended purpose, and rehabilitation work required in the fulfillment of the responsibility of the Corps of Engineers. In addition to the periodic inspections, the structures should be inspected promptly after hurricanes, tsunamis, or other severe storms and floods (ER 1165-2-304). Procedures for inspection and the establishment of an evaluation program can be derived from ER 1110-2-100, Appendix A.

12-4. Monitoring Projects. The objectives, results, and benefits for several of the breakwater or jetty projects in the MCCP program are presented below.

a. Cleveland Harbor, Ohio. The eastern-most 4,400 feet of the Cleveland Harbor breakwater were rehabilitated with 2-ton unreinforced dolosse to ensure the integrity of the 90-year old structure. Work was completed in 1980 (figure 12-1).

(1) Monitoring program objectives.

(a) Quantify armor unit breakage in a structure protected with

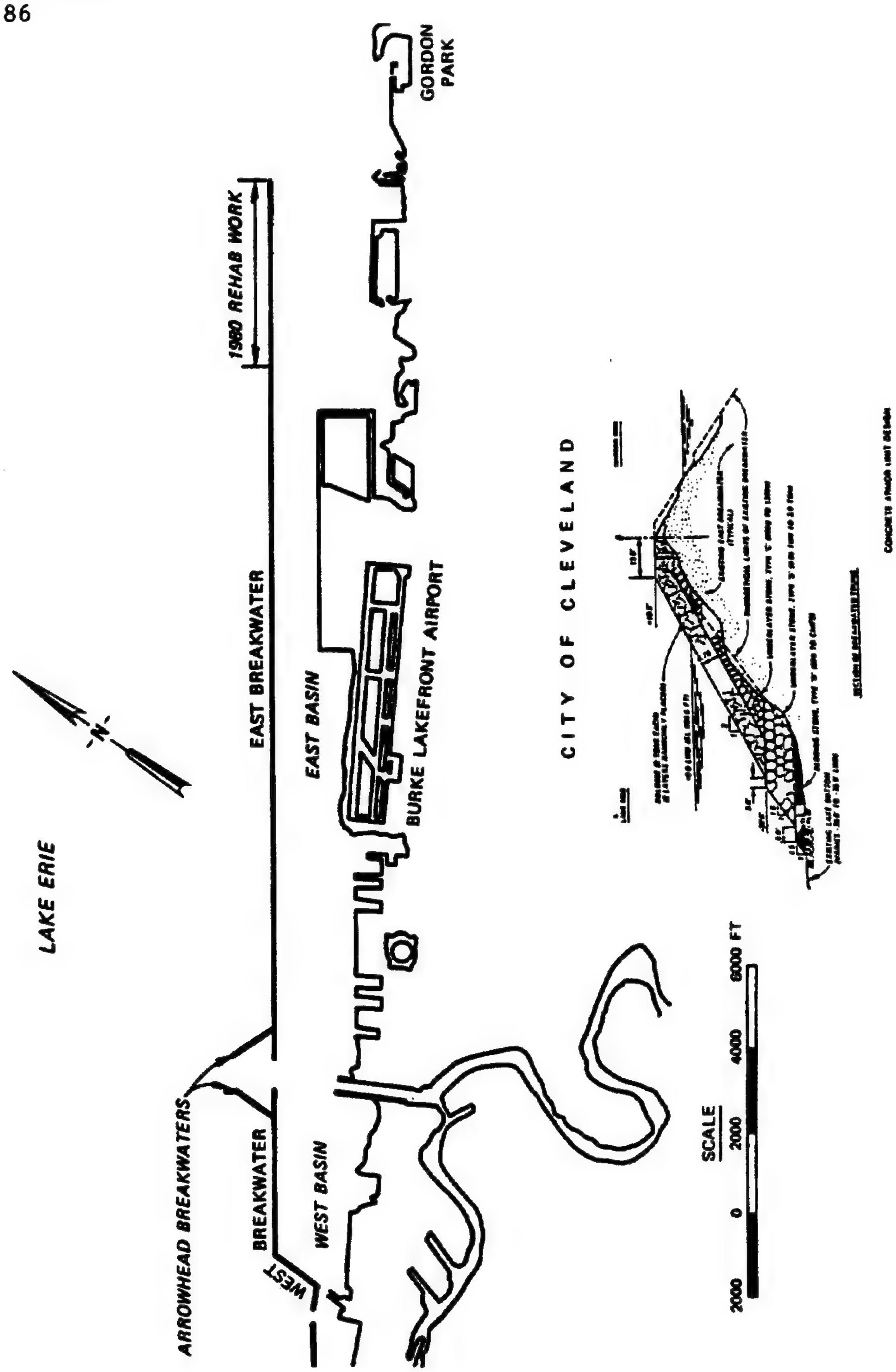


Figure 12-1. Cleveland Harbor, Ohio

unreinforced dolosse and determine the level of breakage that would compromise the integrity of the structure.

(b) Investigate wave transmission by the overtopping of the structure.

(c) Identify effects of ice on the structure.

(d) Evaluate side-scan sonar techniques as an inspection tool for coastal structures.

(2) Results.

(a) Comparison of consecutive sets of aerial photo enlargements (scale 1 inch = 10 feet) was useful in the qualitative monitoring of movement of individual armor units.

(b) An automated procedure for documenting total dolosse movement (defined by vectors for individual dolos) has been developed. The data collected indicate that the dolosse are continuing to settle but the rate of settlement is decreasing.

(c) The computer program developed in FY 82 to maintain an inventory of broken dolosse resulted in quick access to raw data and will expedite determination of statistical patterns of breakage and may suggest predominant mechanisms influencing breakage. The total number of broken dolosse as of May 1983 was 541, or approximately 5.5 percent of the dolosse placed above the water level.

(d) In April 1983, the Buffalo District received an analysis of wave data collected during the ice-free season of 1981, which was prepared in support of the Cleveland Harbor Deep Draft Navigation Study. The comparison of results from CERC's newly developed shallow-water wave hindcast model with field data demonstrated that it can accurately describe time-varying storm wave conditions in spectral form.

(e) While only 60 dolosse were required to repair the damaged head section, 200 were used for overbuilding, essentially adding a third armor layer. These new dolosse are distinctively marked and will be closely monitored. The minor breakage and relative movement between the fall 1982 and spring 1983 surveys may be the result of either the lack of winter lake ice or the general stabilization of the armor units. Continued monitoring will help in determining structural stability and/or if ice is the dominant mechanism producing settlement and breakage.

(f) In the spring of 1983, 35 dolosse were placed at various locations along the structure trunk to repair areas with notable loss of crest height because of breakage or settling. This was the first maintenance of the trunk section since construction was completed.

(3) Monitoring benefits:

(a) The data base acquired by monitoring settlement and breakage of dolosse armor units will contribute to a more complete understanding of the dynamics in dolosse armor layers. The results have already assisted the Buffalo District in developing a more effective maintenance program and will assist the development of maintenance programs for similar structures. These data also will be used to assist in the design of the next phase of rehabilitation at Cleveland Harbor and the evaluation of general design guidance.

(b) Side-scan sonar imagery was demonstrated to be an efficient, quantitative, and cost-effective subsurface inspection technique for coastal structures. While side-scan sonar is particularly useful in turbid water where visual or video inspections are impossible, it can also significantly reduce the cost of inspections in clear water by identifying specific areas of a structure that require more detailed inspection. Although side-scan sonar cannot define individual units, such as dolos within an armor layer, in some cases the necessity for diving operations and video recording can be minimized or eliminated.

(c) To evaluate the design guidance, measurements of wave transmission by overtopping will be compared with the predicted performance. Cleveland Harbor is an ideal location for this comparison since the breakwater is impermeable.

(d) Lake Erie was virtually ice-free during the winter of 1982-83. During that period, settlement and breakage of dolosse greatly diminished as a result of stabilization of the structure, lack of ice, or both. The final year of monitoring should produce valuable data to discriminate the respective effects presuming significant amounts of ice form this winter.

b. Cattaraugus Creek Harbor, New York. Construction of Cattaraugus Creek Harbor, consisting of two shore-connected, rubble-mound breakwaters and nearly one mile of channel improvements, was completed in January 1983 (figure 12-2). The objectives were to maintain the navigation channel and eliminate the bar at the stream mouth, thus minimizing spring ice jams and the resulting floods.

(1) Monitoring program objectives.

(a) Evaluate response of the shoreline and navigation channel to the breakwaters.

(b) Evaluate stability of the rubble-mound structures.

(c) Compare pre- and post-construction sediment budgets.

(d) Investigate onshore/offshore sediment transport near the breakwaters.

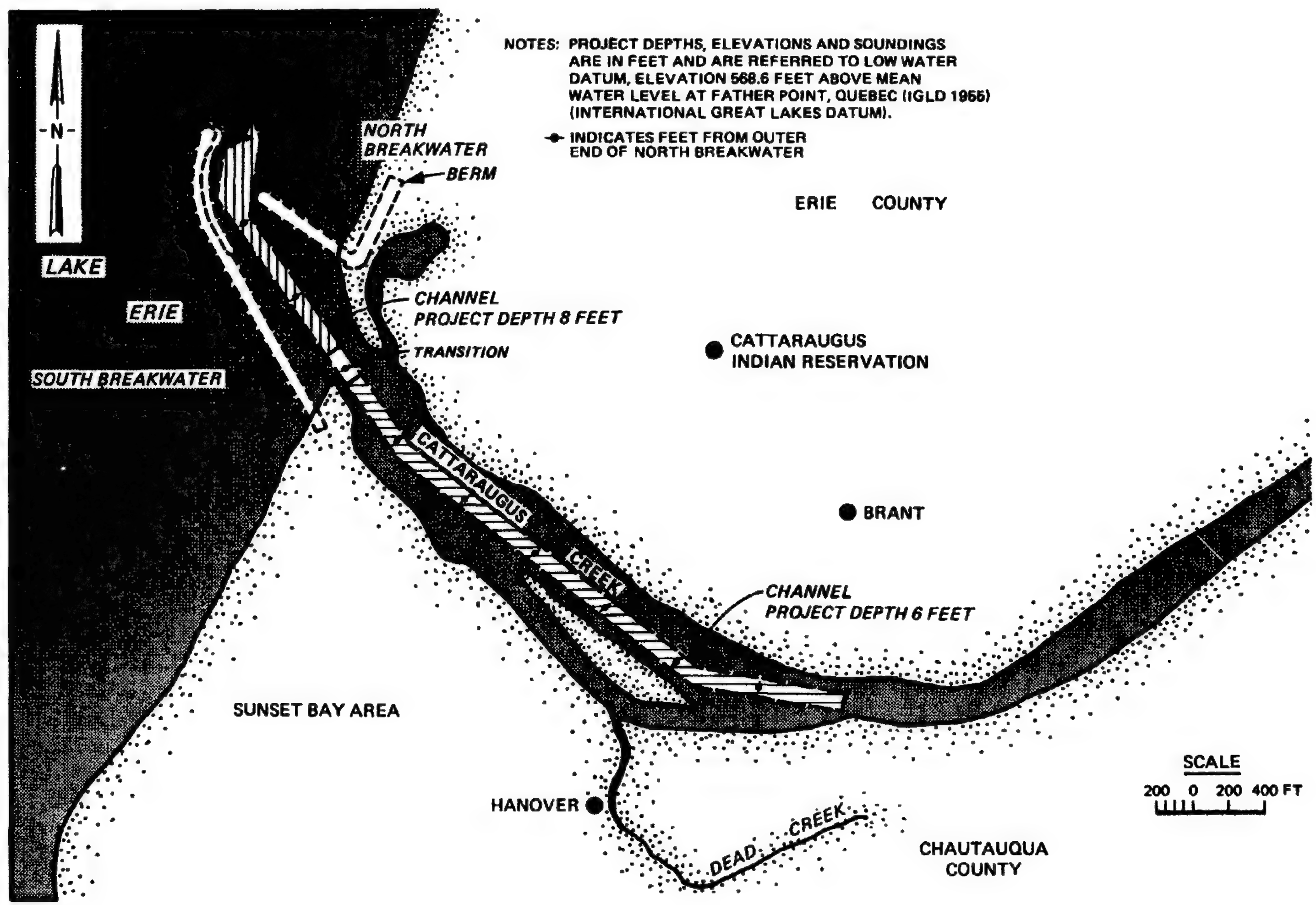


Figure 12-2. Chattaraugus Harbor, New York

(e) Determine effects of ice on the structures and the beach and determine the effect of the structures on ice flow.

(2) Results. Data collected are presently being analyzed. Since the program began in 1983, there are no results to report.

(3) Anticipated monitoring benefits.

(a) Monitoring at Cattaraugus Creek Harbor will provide valuable data delineating the ability of jetties to stabilize a navigation channel in a stream mouth. Field data are critical to verification and/or improvement of design procedures for complicated flow regimes in which sediment transport is driven by both stream and wave effects but is not influenced by tides.

(b) The performance of the structures particularly with respect to eliminating the stream mouth bar (thus minimizing the spring ice jams) will be valuable in the planning and designing of similar projects, particularly in the Great Lakes.

(c) Acquisition of data quantifying structural stability and its effect on sediment transport will support the overall objectives of the MCCP program by evaluating existing design techniques and identifying potential refinements that produce more cost-effective structures.

c. Manasquan Inlet, New Jersey. Jetties at Manasquan Inlet, originally constructed between 1930 and 1931, were rehabilitated with 16-ton reinforced concrete dolosse. Rehabilitation of the south jetty occurred between 1979 and 1980; the north jetty was rehabilitated between 1981 and 1982 (figure 12-3).

(1) Monitoring program objectives:

(a) Evaluate preformance of the dolosse armor units in retaining the structural stability of the jetty.

(b) Determine potential effects on longshore transport in the vicinity of the inlet.

(c) Evaluate effectiveness of rehabilitated jetties in maintaining a stable inlet channel cross section.

(2) Results (1 July 1982 to 30 June 1983).

(a) The LEO observor has attained a 95 percent completion rate for making twice daily observations and obtained the only wave height estimates for 2-30 March 1982 and 18 February-15 March 1983 when the buoy was not in operation.

(b) During a significant storm in October 1982 which resulted in the

Figure 12-3. Manasquan Inlet, New Jersey

loss of the wave buoy, a significant wave height of 9 meters (29.5 feet) was recorded. Minimum tide heights during that storm averaged 1.2 feet above normal.

(c) Tidal prism measurements indicate a flood flow of $3.80X10^8$ cubic feet and an ebb flow of $3.08X10^8$ cubic feet. It is important to note that these results agree favorably with Jarrett's equation. Using the measured inlet cross-section area (6,822 square feet) in Jarrett's tidal prism-inlet area equation for dual-jettied Atlantic Coast inlets, the predicted tidal prism is $3.08X10^8$ cubic feet.

(d) Survey of the submerged portion of the south jetty with side-scan sonar was unsuccessful in discriminating dolosse from armor stone. Fixed mounting of the sonar to the boat and adverse wave conditions resulted in the reproduction (or superposition) of wave motion on the imagery.

(e) The inlet hydrographic survey indicates that the jetty improvements are not yet maintaining the navigation channel as designed. This is not surprising since the survey was made before completion of the north jetty improvements.

(f) Beach profiles taken before and after the October 1982 storm have documented the response of the beach to the storm. An average of 6 cubic yards of sand were lost per linear foot of beach above the -2.0 feet NGVD contour, although localized effects ranged from slight accretion to as much as 20 cubic feet of erosion per lineal foot of beach.

(g) Photogrammetric measurements of dolosse movement compared favorably with standard leveling techniques, i.e. ± 0.2 foot. A minor, nonlocalized settlement of the south jetty has occurred. Of the dolosse that have settled, 90 percent of the downward motion was 0.3 foot or less and only one dolosse moved more than 1.0 foot (1.5 feet).

(3) Monitoring benefits.

(a) Additional guidance on executing side-scan sonar surveys was developed. Although side-scan sonar imagery is an effective method of evaluating the integrity of coastal structures, it should not be indiscriminately used in shallow water, especially if significant waves exist. In shallow water, susceptibility of the sonar "fish" to damage on the bottom requires a short scope in the towline which may result in the reproduction of wave-induced ship motions in the record and produce distortion of the imagery.

(b) Comparisons of photogrammetric and standard leveling techniques for measuring dolosse movement on the jetties demonstrated better agreement than anticipated. Photogrammetric techniques have more than adequate accuracy for a myriad of coastal engineering applications and, in numerous instances, significant savings can be accrued by using photogrammetric mapping as compared to "conventional" leveling.

(c) Data quantifying settlement and breakage of the dolosse armor units will contribute to identification of the dominant mechanisms that produce failure of dolosse armor layers. This information (as well as data from Cleveland Harbor) will assist in the development of a maintenance program for this and similar projects. These data will also be used to evaluate the existing design guidance.

(d) Measurements of the tidal prism have been compared to and agree favorably with Jarrett's tidal prism-inlet area equation. If subsequent data verify the preliminary findings, the increased confidence in Jarrett's equation would result in a reduction in expensive inlet flow measurements.

d. Umpqua River, Oregon. In 1977, improvements to the north jetty at the Umpqua River entrance were undertaken to reduce the amount of sediment passing through the structure and to reduce shoaling in the channel. An extension of the training jetty to connect with the south jetty was completed in 1980 to further stabilize the channel and reduce reported cross currents (figure 12-4).

(1) Monitoring program objectives.

(a) Compare present prototype conditions to those predicted by previous studies by evaluating response of river mouth, navigation channel, and beach to the jetty improvements.

(b) Evaluate wave transformation characteristics from deep water to the project site.

(c) Correlate nearshore wave conditions with structural damage.

(2) Results 1 July 1982 to 30 June 1983.

(a) A surface float study was performed in the river mouth. Traces of float movement were compared with photographs of styrofoam chip movement in the physical model.

(b) All other data are presently being analyzed, so no results were available for this manual.

(3) Anticipated monitoring benefits. Data collected will be used to compare prototype response with responses predicted by design guidelines and by a physical model. Wave data will be obtained offshore of and within the harbor to evaluate the transformation of waves as they propagate into the harbor and nearshore wave conditions will be correlated to structural damage. Improved guidance in each of these areas will result.

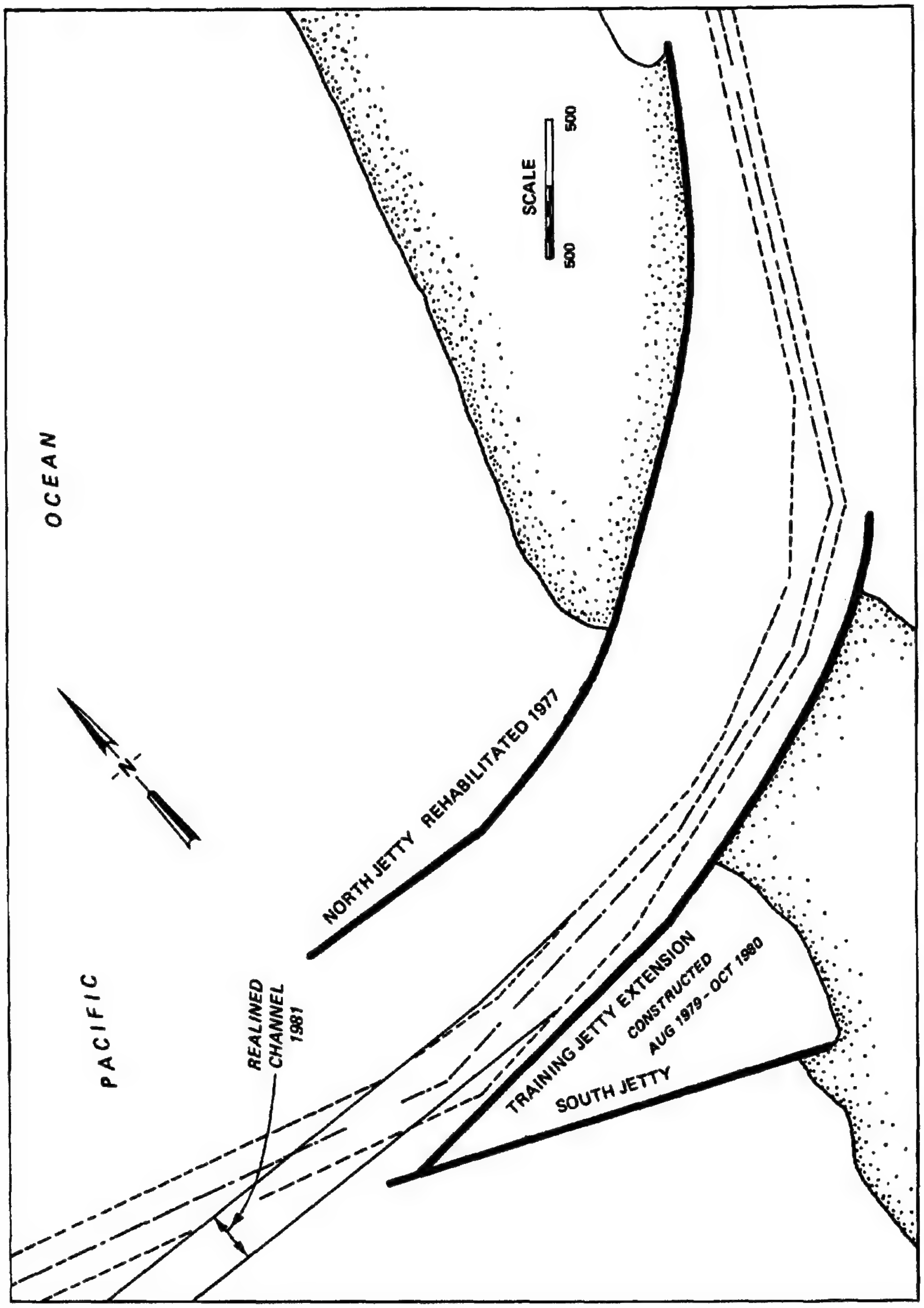


Figure 12-4. Umpqua River, Oregon

APPENDIX A

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APPENDIX B

INVENTORY OF WATERWAYS EXPERIMENT STATION MODEL TESTS

B-1. General. Numerous breakwater and jetty model investigations have been conducted at the US Army Engineer Waterways Experiment Station (WES). These investigations, which are summarized in the following paragraphs, should provide excellent guidance as to the type of design information obtainable from these studies. Table B-1 lists pertinent information from each model study and shows the variance of the stability coefficient for different types of armor and environmental conditions used on similar types of structures.

B-2. Stability Tests Conducted on Breakwater or Jetty Trunk Sections (New Construction).

a. Purposes of Studies. The purposes of these studies are typically to experimentally investigate through two-dimensional model tests the armor stability, wave transmission properties, and wave overtopping characteristics of a proposed breakwater trunk section.

b. Tests and Results. Tests are conducted using the range of water levels, wave periods, and wave heights that are expected during the design life of the structure. Alternate plans that may reduce the structure's cost without significantly affecting its performance are investigated if the original section proves to be acceptable. If the original section proves to be inadequate, modifications are made as needed to achieve an acceptable level of stability and wave protection. Thus the model serves as a tool to aid in optimization of the structure.

c. Studies Conducted.

(1) Waianae Small-Boat Harbor, Oahu, Hawaii, Design for Wave Protection (item 8).

(2) Stability of Rubble-Mound Breakwater, Lahaina Harbor, Hawaii (item 22).

(3) Rubble-Mound Breakwater Stability and Wave-Attenuation Tests, Port Ontario Harbor, New York (item 34).

(4) Stability of Rubble-Mound Breakwater, Maalaea Harbor, Maui, Hawaii (item 33).

(5) South Jetty Stability Study, Masonboro Inlet, North Carolina (item 32).

(6) Designs for Rubble-Mound Breakwaters, Dana Point Harbor, California (item 42).

(7) Stability and Transmission Tests of Tribar Breakwater Section Proposed for Monterey Harbor, California (item 44).

(8) Stability Tests on Proposed Rubble-Mound Breakwaters, Nassau Harbor, Bahamas (item 67).

(9) Design of Tetrapod Cover Layer for a Rubble-Mound Breakwater, Crescent City Harbor, Crescent City, California (item 64).

(10) Stability of Crescent City Harbor Breakwater, Crescent City, California (item 54).

(11) Stability of Proposed Breakwater, Burns Waterway Harbor, Indiana (item 75).

(12) Designs for Rubble-Mound Breakwater, Noyo Harbor, California (item 74).

(13) Placed-Stone Stability Tests, Tillamook, Oregon (item 91).

B-3. Stability Tests Conducted on Breakwater or Jetty Head and Trunk Sections (New Construction).

a. Purposes of Studies. The purposes of these types of studies are to experimentally investigate through three-dimensional model tests the armor stability, wave transmission properties, and wave overtopping characteristics of a proposed breakwater trunk and head section.

b. Tests and Results. Tests are typically the same as those described in paragraph B-2b except that they are conducted for at least two angles of wave attack. Again, test results are used to aid in optimization of the structure.

c. Studies Conducted.

(1) Jetty Stability Study, Oregon Inlet, North Carolina (item 31).

(2) Stability of Rubble-Mound Breakwaters, Jubail Harbor, Saudi Arabia (item 29).

(3) Designs for Rubble-Mound Breakwater Construction, Tsoying Harbor, Taiwan (item 71).

(4) Breakwater Stability Study, Mission Bay, California (item 89).

(5) Breakwater and Revetment Stability Study, San Juan National Historic Site, San Juan, Puerto Rico (item 86).

(6) Breakwater Stability Study, Imperial Beach, California (item 90).

B-4. Stability Tests Conducted on Breakwater or Jetty Sections for Rehabilitation and/or Repair of Existing Structures.

a. Purposes of Studies. Studies of this type typically investigate the adequacy of proposed repair plans and, if necessary, develop alternate designs from which the optimum plan for stability, constructability, and economy can be determined.

b. Tests and Results. Structures in need of repair or rehabilitation have normally been subjected to wave conditions in excess of the originally estimated design conditions. Thus, model tests typically simulate those storm conditions that have produced damage to the prototype structure.

c. Studies Conducted.

(1) Stability Tests of Modified Repair Options for the San Pedro Breakwater, Los Angeles, California (item 7).

(2) Breakwater Rehabilitation Study, Crescent City Harbor, California (item 6).

(3) San Pedro Breakwater Repair Study, Los Angeles, California (item 28).

(4) Stability Tests of Nawiliwili Breakwater Repair (item 47).

(5) Proposed Jetty-Head Repair Sections, Humboldt Bay, California (item 46).

(6) Designs for Rubble-Mound Breakwater Repair, Kahului Harbor, Maui, Hawaii (item 72).

(7) Designs for Rubble-Mound Breakwater Repair, Morro Bay Harbor, California (item 70).

(8) Design for Rubble-Mound Breakwater Repairs, Nawiliwili Harbor, Nawiliwili, Hawaii (item 77).

(9) Kahului Breakwater Stability Study, Kahului, Maui, Hawaii (item 87).

(10) Nawiliwili Breakwater Stability Study, Nawiliwili Harbor, Kauai, Hawaii (item 92).

Table B-1. Breakwater Stability Two- and Three-Dimensional Site-Specific Model Studies Conducted at US Army Engineer Waterways Experiment Station

Project Location	Type of Structure	Type of Armor	Type of Placement	Armor weight tons	Armor Specific Weight lb/ft^3	Sea-bottom Slope	Sea-side Structure Slope	Total Water Depth ft	Structure Height ft	Degree of Overtop	Wave Period sec	Wave Height ft	Model K_D	Wave Form	Angle of Wave Attack deg	Reference[a]
						Nonbreaking Waves on Breakwater Trunks										
Rough Angular Stone																
Burns Harbor, Indiana[b]	Breakwater	Stone	Random 2 Layers	13.5	165	1:100	1:1.5	47.0	57.0	Mod	11	15.0	3.1	Non-breaking	90.0	TR 2-766 Mar 1967 (item 75)
Long Beach (Sohio), California[b]	Breakwater	Stone	Random 2 Layers	7.5	165	Flat	1:2	55.4	64.0	Min	13	14.0	3.8	Non-breaking	90.0	Unpublished[c]
Mission Bay, California[b]	Breakwater	Stone	Random 2 Layers	14.5	165	Flat	1:1.5	34.6	47.5	Mod	9-15	16.7	4.3	Non-breaking	90.0	TR HL-83-18 Sep 1983 (item 89)
Morro Bay, California[d]	Breakwater	Stone	Random 2 Layers	25.0	175	1:50	1:2.25	38.0	48.0	Maj	15	27.0	5.6	Non-breaking	90.0	TR 2-567 May 1961 (item 70)
												28.0	6.4	Non-breaking		
Rough Dimensional Stone (Special Placed)																
Dana Point, California[b]	Breakwater	Stone	Special 2 Layers	12.0	165	1:30	1:2	37.0	44.0	Maj	18	22.0	9.3	Non-breaking	90.0	TR 2-725 Jun 1966 (item 42)
Siuslaw, Oregon[d]	Breakwater	Stone	Special 2 Layers	10.0	170	Flat	1:2	30.0	42.0	Mod	12	17.0	4.6	Non-breaking	90.0	TR 2-631 July 1963 (item 135)
	Breakwater	Stone	Special 1 Layer	10.0	170	Flat	1:1.5	30.0	40.0	Mod	12	15.0	4.2	Non-breaking	90.0	
	Breakwater	Stone	Special 2 Layers	10.0	170	Flat	1:2	30.0	42.0	Mod	12	19.0	6.4	Non-breaking	90.0	
	Breakwater	Stone	Special 2 Layers	10.0	170	Flat	1:2	30.0	49.5	Minor	12	16.0	4.4	Non-breaking	90.0	
	Breakwater	Stone	Special 1 Layer	10.0	170	Flat	1:1.5	30.0	47.5	Minor	12	15.0	4.2	Non-breaking	90.0	
	Breakwater	Stone	Special 2 Layers	10.0	170	Flat	1:2	30.0	49.5	Minor	12	17.0	5.2	Non-breaking	90.0	

(Continued)

[a] All references refer to WES publications.

[b] New projects.

[c] "Long Beach-Sohio Breakwater Stability Study: Long Beach, California," by R. D. Carver, 1979 (can be obtained from Port of Long Beach, Long Beach, California 90801).

[d] Rehabilitated projects.

(Sheet 1 of 8)

Table B-1 (Continued)

Project Location	Type of Structure	Type of Armor	Type of Placement	Armor weight tons	Armor Specific Weight lb/ft^3	Sea-bottom Slope	Sea-side Structure Slope	Total Water Depth ft	Structure Height ft	Degree of Overtop	Wave Period sec	Wave Height ft	Model K_D	Wave Form	Angle of Wave Attack deg	Reference
					Nonbreaking Waves on Breakwater Trunks (Continued)											
Tetrapods/Quadripods																
Crescent City, California[a]	Breakwater	Tetrapod	Random	17.6	140	Flat	1:2	69.0	84.0	Mod	14	23.0	14.4	Non-breaking	90.0	TM 2-413 Jun 1955 (item 64)
	Breakwater	Tetrapod	Random	17.6	140	Flat	1:3	69.0	84.0	Mod	14	25.0	12.4	Non-breaking	90.0	
	Breakwater	Tetrapod	Random	17.6	140	Flat	1:4	69.0	84.0	Mod	14	26.0	10.4	Non-breaking	90.0	
Crescent City, California[a]	Breakwater	Tetrapod	Random	25.0	150	Flat	1:1.333	55.0	70.0	Mod	14	20.0	7.4	Non-breaking	90.0	MP 2-171 Apr 1956 (item 65)
Tribars																
Monterey Harbor, California[a]	Breakwater	Tribars	Random 2 Layers	12.0	150	Flat	1:1.5	45.2	54.0	Maj	17	18.0	10.0	Non-breaking	90.0	MP H-69-11 Sep 1969 (item 44)
Morro Bay, California[b]	Breakwater	Tribars	Random 1 Layer	20.0	150	1:50	1:1.5	38.0	48.0	Maj	13	24.0	14.4	Non-breaking	90.0	TR 2-567 May 1961 (item 70)
						Breaking Waves on Breakwater Trunks										
Rough Angular Stone																
Lahaina Harbor, Hawaii[a]	Breakwater	Stone	Random 2 Layers	2.75	170	1:20	1:2	7.5	17.0	None	16	8.0	1.8	Breaking	90.0	TR H-76-8 Apr 1976 (item 22)
Jubail Harbor, Saudi Arabia[a]	Breakwater	Stone	Random 2 Layers	7.15	165	1:10	1:2	19.7	36.1	None	9	13.4	3.5	Breaking	90.0	TR H-76-20 Dec 1976 (item 29)
Port Ontario Harbor, New York[c]	Breakwater	Stone	Random 2 Layers	5.3	155	1:50	1:2	12.3	17.7	Mod	11	9.8	2.1	Breaking	90.0	TR HL-81-5 Apr 1981 (item 34)
San Juan, Puerto Rico[c]	Breakwater	Stone	Random 2 Layers	33.9	165	1:20	1:2	22.9	23.0	Sev	17	23.3	3.9	Breaking	90.0	TR HL-81-11 Sep 1981 (item 86)

(Continued)

(a) New projects.
(b) Rehabilitated projects.

(Sheet 2 of 8)

Table B-1 (Continued)

Project Location	Type of Structure	Type of Armor	Type of Placement	Armor weight tons	Armor Specific Weight lb/ft^3	Sea-bottom Slope	Sea-side Structure Slope	Total Water Depth ft	Structure Height ft	Degree of Overtop	Wave Period sec	Wave Height ft	Model K_D	Wave Form	Angle of Wave Attack deg	Reference
Breaking Waves on Breakwater Trunks (Continued)																
Rough Angular Stone (Cont)																
Masonboro Inlet, North Carolina[a]	Jetty	Stone	Random 2 Layers	18.0	165	1:20	1:2	14.5	13.0	Sev	15	13.5	1.4	Breaking	90.0	MP H-78-12 Oct 1978 (item 32)
Oregon Inlet, North Carolina[a]	Jetty	Stone	Random 2 Layers	22.0	165	1:20	1:1.5	16.5	22.5	Sev	15	15.5	2.4	Breaking	90.0	TR CERC-83-3 Sep 1983 (item 31)
Fort Fisher, North Carolina[a]	Revetment	Stone	Random 2 Layers	4.3	165	1:55	1:2	14.7	17.0	Mod-Maj	10	11.8	4.0	Breaking	90.0	TR HL-82-26 Nov 1982 (item 38)
Rough Dimensioned Stone (Special Placed)																
Tillamook, Oregon[b]	Breakwater	Stone	Special 2 Layers	9.5	170	1:20	1:2	30.0	40.0	Mod	13	26.0	17.2	Breaking	90.0	TR HL-79-16 Sep 1979 (item 91)
	Toe	Stone	Special 2 Layers	6.4	170	1:20	1:1.5	20.0	40.0	Min	13	16.0	7.9	Breaking	90.0	
	Breakwater	Stone	Special 2 Layers	22.6	170	1:20	1:2	40.0	50.0	Mod	13	33.0	14.9	Breaking	90.0	
	Toe	Stone	Special 2 Layers	15.2	170	1:20	1:1.5	30.0	50.0	Min	13	26.0	14.5	Breaking	90.0	
	Breakwater	Stone	Random 2 Layers	22.6	170	1:20	1:2	40.0	50.0	Mod	13	33.0	22.2	Breaking	90.0	
	Toe	Stone	Random	15.2	170	1:20	1:2	30.0	50.0	Min	13	26.0	7.3	Breaking	90.0	
Fort Fisher, North Carolina[a]	Revetment	Stone	Special 2 Layers	3.0	165	1:55	1:2	14.7	17.0	Non-Maj	10	11.8	5.8	Breaking Nov 82	90.0	TR HL-82-26 Nov 82 (item 88)
Tetrapods/Quadripods																
Noyo Harbor, California[a]	Breakwater	Tetrapod	Random	36.0	150		1:3	48.0	60.0	Mod	14	29.0	7.0	Breaking	90.0	MP 2-841 Aug 1966 (item 74)

(Continued)

[a] New projects.
[b] Rehabilitated projects.

(Sheet 3 of 8)

Table B-1 (Continued)

Project Location	Type of Structure	Type of Armor	Type of Placement	Armor weight tons	Armor Specific Weight lb/ft^3	Sea-bottom Slope	Sea-side Structure Slope	Total Water Depth ft	Structure Height ft	Degree of Overtop	Wave Period sec	Wave Height ft	Model K_D	Wave Form	Angle of Wave Attack deg	Reference
Breaking Waves on Breakwater Trunks (Continued)																
Tribars																
Nassau Harbor, Bahamas[a]	Breakwater	Tribars	Random 2 Layers	10.0	150	Flat	1:1.5	25.0	35.0	Mod/Maj	11	19.0	14.1	Breaking	90.0	MP 2-799 Mar 1966 (item 67)
				19.0	150	Flat	1:1.5	30.0	40.0	Maj	11	23.0	13.2	Breaking	90.0	
	Breakwater	Tribars	Random 2 Layers	31.0	150	Flat	1:1.5	38.0	48.0	Maj	11	25.0	10.4	Breaking	90.0	
Kahului Harbor, Maui, Hawaii[b]	Breakwater	Tribars	Random 2 Layers	35.0	156	1:125	1:2	58.0	68.0	Sev	18	37.0	19.0	Breaking	90.0	TR 2-644 Feb 1964 (item 72)
Kahului, Maui, Hawaii[b]	Breakwater	Tribars	Random 2 Layers	35.0	146	1:27	1:3.6	42.0	55.8	Maj	18	34.0	10.6	Breaking	90.0	TR HL-82-14 Jul 1982 (item 87)
Kahului, Maui, Hawaii[b]	Breakwater	Tribars	Random 2 Layers	19.0	146	1:27	1:2.6	29.0	41.3	Maj	18	25.6	11.8	Breaking	90.0	
Kahului, Maui, Hawaii[b]	Breakwater	Tribars	Special 1 Layer	10.0	146	1:27	1:2	24.0	32.1	Maj	18	21.5	15.7	Breaking	90.0	
Nawiliwili Harbor, Hawaii[b]	Breakwater	Tribars	Special 1 Layer	17.8	158	1:55	1:1.5	32.0	45.0	Maj	16	24.0	12.9	Breaking	90.0	MP 2-377 Feb 1960 (item 77)
Dolos																
Lahaina Harbor, Hawaii[a]	Breakwater	Dolos	Random 2 Layers	0.75	150	1:20	1:2	7.5	14.5	None	16	8.0	10.5	Breaking	90.0	TR H-76-8 Apr 1976 (item 22)
Maalaea Harbor, Maui, Hawaii[a]	Breakwater	Dolos	Random 2 Layers	6.0	147	1:50	1:1.5	19.0	28.0	Maj	16	16.7	17.5	Breaking	90.0	MP HL-81-1 Jan 1981 (item 33)
Waianae Harbor, Oahu, Hawaii[a]	Breakwater	Dolos	Random 2 Layers	1.5	150	1:20/ 1:53	1:2	16.0	28.0	Mod	16	11.8	16.9	Breaking	90.0	TR H-76-8 May 1976 (item 8)

(Continued)

[a] New projects.
[b] Rehabilitated projects.

(Sheet 4 of 8)

Table B-1 (Continued)

Project Location	Type of Structure	Type of Armor	Type of Placement	Armor weight tons	Armor Specific Weight lb/ft^3	Sea-bottom Slope	Sea-side Structure Slope	Total Water Depth ft	Structure Height ft	Degree of Overtop	Wave Period sec	Wave Height ft	Model K_D	Wave Form	Angle of Wave Attack deg	Reference
Breaking Waves on Breakwater Trunks (Continued)																
Dolos (Continued)																
Kahului, Maui, Hawaii[a]	Breakwater	Dolos	Random 2 Layers	30.0	146	1:100	1:1.7	49.0	56.5	Maj	18	29.8	18.3	Breaking	90.0	TR HL-82-14 Jul 1982 (item 87)
Nawiliwili Harbor, Hawaii[a]	Breakwater	Dolos	Random 2 Layers on toe	2.0	146	1:10	1:1.5	10.0	16.5	Min	16	8.9	8.2	Breaking	90.0	MP H-78-4 Jan 1978 (item 47)
Oregon Inlet, North Carolina[b]	Jetty	Dolos	Random 2 Layers	9.5	150	1:20	1:1.5	16.5	22.5	Sev	15	15.5	8.3	Breaking	90.0	TR CERC-83-3 Sep 1983 (item 31)
Atlantic Station, New Jersey[b]	Breakwater	Dolos	Random 2 Layers	43.0	150	1:10	1:2	56.3	104.0	Min-Mod	16	40.0	23.0	Breaking	90.0	Unpublished[c]
Nonbreaking Waves on Breakwater Heads																
Rough Angular Stone																
Mission Bay, California[b]	Breakwater	Stone	Random 2 Layers	14.5	165	Flat	1:2	34.6	47.5	Mod	9-15	16.7	3.4	Non-breaking	33.0	TR HL-83-18 Sep 1983 (item 89)
Oregon Inlet, North Carolina[b]	Jetty	Stone	Random 2 Layers	30.0	165	1:20	1:3	28.0	38.0	Mod	15	17.6	1.3	Non-breaking	90.0	TR CERC-83-3 Sep 1983 (item 31)
								28.0	38.0	Maj	15	22.0	2.5	Non-breaking	45.0	
Rough Dimensioned Stone (Special Placed)																
Siuslaw, Oregon[a]	Breakwater	Stone	Special 3 Layers	18.0	170	Flat	1:2	40.0	51.5	Mod	14	21.0	4.8	Non-breaking	90.0	TR 2-631 Jul 1963 (item 135)
				18.0	170	Flat	1:2	40.0	51.5	Mod	14	21.0	7.2		90.0	
				18.0	170	Flat	1:2	40.0	51.5	Mod	14	24.0	7.2		90.0	
				18.0	170	Flat	1:1.5	40.0	59.0	Minor	14	24.0	8.2		90.0	
				18.0	170	Flat	1:1.5	40.0	59.0	Minor	14	22.0	6.3		90.0	
				18.0	170	Flat	1:1.5	40.0	59.0	Minor	14	24.0	8.2		90.0	

(Continued)

(a) Rehabilitated projects.
(b) New projects.
(c) "Atlantic Generating Station Breakwater Stability Study" by D. D. Davidson and D. G. Markle, 1978 (can be obtained from Public Service Electric and Gas Co. of New Jersey, Newark, New Jersey 07101).

(Sheet 5 of 8)

Table B-1 (Continued)

Project Location	Type of Structure	Type of Armor	Type of Placement	Armor weight tons	Armor Specific Weight lb/ft^3	Sea-bottom Slope	Sea-side Structure Slope	Total Water Depth ft	Structure Height ft	Degree of Overtop	Wave Period sec	Wave Height ft	Model K_D	Wave Form	Angle of Wave Attack deg	Reference
					Nonbreaking Waves on Breakwater Heads (Continued)											
Rough Dimensioned Stone (Special Placed) (Continued)																
Tetrapods/Quadripods																
Nassau Harbor, Bahamas[a]	Breakwater	Quadrapod	Random	7.0	150	Flat	1:1.5	25.0	35.0	Min	11	12.0	5.1	Non-breaking	90.0	TR 2-697 Oct 1965 (item 73)
	Breakwater	Tetrapod	Random	14.0	150	Flat	1:1.5	25.0	35.0	Mod	11	15.0	5.0	Non-breaking	90.0	
	Breakwater	Tetrapod	Random	17.0	150	Flat	1:1.5	25.0	35.0	Mod	11	16.0	5.0	Non-breaking	90.0	
Humboldt Bay, California[b]	Jetty	Tetrapod	Random	28.0	150	1:10	1:5	43.0	60.0	Min	16	23.0	2.7	Non-breaking	90.0	
Tribars																
Humboldt Bay, California[b]	Jetty	Tribars	Random 2 Layers	23.0	150	1:10	1:5	43.0	60.0	Mod	16	29.0	6.6	Non-breaking	45.0	TR HL-71-8 Nov 1971 (item 46)
	Jetty	Tribars	Random 2 Layers	33.0	150	1:10	1:5	43.0	60.0	Mod	16	36.0	6.6	Non-breaking	45.0	
	Jetty	Tribars	Random 2 Layers	44.0	150	1:10	1:5	43.0	60.0	Maj	16	36.0	6.6	Non-breaking	45.0	
Kahului Harbor, Maui, Hawaii[b]	Breakwater	Tribars	Random 2 Layers	35.0	156	1:125	1:3	58.0	68.5	Maj	14	30.0	6.8	Non-breaking	90.0	TR 2-644 Feb 1964 (item 72)
	Breakwater	Tribars	Random 2 Layers	50.0	156	1:125	1:4	58.0	85.5	Min	18	35.0	5.6	Non-breaking	90.0	
	Breakwater	Tribars	Random 2 Layers	35.0	156	1:125	1:3	58.0	68.5	Maj	18	36.0	8.2	Non-breaking	50.0	
	Breakwater	Tribars	Random 2 Layers	50.0	156	1:125	1:3	58.0	68.5	Maj	18	36.0	8.2	Non-breaking	50.0	
Morro Bay, Harbor California[b]	Breakwater	Tribars	Random 1 Layer Above swl 2 Layers Below swl	20.0	150	1:50	1:2.25	38.0	48.0	Maj	15	25.0	10.7	Non-breaking	90.0	TR 2-567 May 1961 (item 70)

(Continued)

(a) New projects.
(b) Rehabilitated projects.

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Table B-1 (Continued)

Project Location	Type of Structure	Type of Armor	Type of Placement	Armor weight tons	Armor Specific Weight lb/ft^3	Sea-bottom Slope	Sea-side Structure Slope	Total Water Depth ft	Structure Height ft	Degree of Overtop	Wave Period sec	Wave Height ft	Model K_D	Wave Form	Angle of Wave Attack deg	Reference
						Nonbreaking Waves on Breakwater Heads (Continued)										
Dolos																
Jubail Harbor, Saudi Arabia[(a)]	Breakwater	Dolos	Random 2 Layers	5.5	150	Flat	1:2	29.5	46.0	None	9	15.4	10.3	Non-breaking	54.0	TR H-76-20 Dec 1976 (item 29)
	Breakwater	Dolos	Random 2 Layers	5.5	150	Flat	1:2	29.5	46.0	None	9	15.4	10.3	Non-breaking	68.0	
Oregon Inlet, North Carolina[(a)]	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	28.0	38.0	Mod	15	17.6	4.0	Non-breaking	0.0	TR CERC-83-3 Sep 1983 (item 31)
	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	28.0	38.0	Mod	15	17.6	4.0	Non-breaking	22.5	
	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	28.0	38.0	Mod	15	17.6	4.0	Non-breaking	45.0	
	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	28.0	38.0	Mod	15	17.6	4.0	Non-breaking	67.5	
	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	28.0	38.0	Mod	15	17.6	4.0	Non-breaking	90.0	
	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	28.0	38.0	Sig	15	22.0	7.8	Non-breaking	45.0	
Cubes																
Humboldt Bay, California[(b)]	Jetty	Cube	Random 1-2 Layers	100.0	150	1:10	1:5	43.0	60.0	Min	16	22.0	0.7	Non-breaking	45.0	TR H-71-8 Nov 1971 (item 46)
Tri-Long																
Humboldt Bay, California[(b)]	Jetty	Tri-Long	Random 1-2 Layers	28.0	150	1:10	1:5	43.0	60.0	Min	16	21.0	2.1	Non-breaking	45.0	TR H-71-8 Nov 1971 (item 46)
						Breaking Waves on Breakwater Heads										
Rough Angular Stone																
Oregon Inlet, North Carolina[(b)]	Jetty	Stone	Random 2 Layers	30.0	165	1:20	1:3	21.0	38.0	Min	15	17.6	1.3	Breaking	90.0	TR CERC-83-3 Sep 1983 (item 31)

(Continued)

[(a)] New projects.
[(b)] Rehabilitated projects.

(Sheet 7 of 8)

Table B-1 (Concluded)

Project Location	Type of Structure	Type of Armor	Type of Placement	Armor weight tons	Armor Specific Weight lb/ft^3	Sea-bottom Slope	Sea-side Structure Slope	Total Water Depth ft	Structure Height ft	Degree of Overtop	Wave Period sec	Wave Height ft	Model K_D	Wave Form	Angle of Wave Attack deg	Reference
					Breaking Waves on Breakwater Heads (Continued)											
Rough Angular Stone (Continued)																
Oregon Inlet, North Carolina[a]	Jetty			30.0	165	1:20	1:3	23.0	38.0	Mod	15	19.2	1.7	Breaking	45.0	
San Juan, Puerto Rico[a]	Breakwater	Stone	Random 2 Layers	27.7	165	1:20	1:2	26.9	27.0	Sev	17	28.0	8.3	Breaking	72.0	TR HL-81-11 Sep 1981 (item 86)
	Breakwater	Stone	Random 2 Layers	27.7	165	1:20	1:2	26.9	27.0	Sev	17	28.0	8.3	Breaking	42.0	
Dolos																
Oregon Inlet, North Carolina[a]	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	21.0	38.0	None	15	17.6	4.0	Breaking	0.0	TR CERC-83-3 Sep 1983 (item 31)
	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	21.0	38.0	None	15	17.6	4.0	Breaking	22.5	
	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	21.0	38.0	None	15	17.6	4.0	Breaking	45.0	
	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	21.0	38.0	None	15	17.6	4.0	Breaking	67.5	
	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	21.0	38.0	None	15	17.6	4.0	Breaking	90.0	
Oregon Inlet, North Carolina[a]	Jetty	Dolos	Random 2 Layers	14.0	150	1:20	1:3	23.0	38.0	Min	15	19.2	5.2	Breaking	45.0	
Humboldt Bay, California[b]	Jetty	Dolos	Random 2 Layers	45.0	155	1:10	1:5	43.0	60.0	Maj	16	40.0	7.7	Breaking	45.0	
Atlantic Station, New Jersey[a]	Breakwater	Dolos	Random 2 Layers	62.0	150	1:10	1:3	56.3	104.0	Min-Mod	16	40.0	10.6	Breaking	Var	Unpublished[c]

(a) New projects.
(b) Rehabilitated projects.
(c) "Atlantic Generating Station Breakwater Stability Study" by D. D. Davidson and D. G. Markle, 1978 (can be obtained from Public Service Electric and Gas Co. of New Jersey, Newark, New Jersey 07101).

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APPENDIX C

NOTATION

Symbol	Term	Units
A	Area covered by armor units	ft^2
B	Crest width	ft
C_r	Coefficient of wave reflection	---
C_t	Transmission coefficient	---
C_u	Undrained cohesive strength of soil	lb/ft^2
d	Water depth	ft
d_1	Submergence of orifices	ft
d/L	Relative depth	---
d_b	Depth of breaking	ft
D	Pile diameter	ft
D_t	Diameter of scrap tire	ft
e	Distance load is applied above the bottom or efficiency	ft ---
F	Force	lb
F_s	Factor of safety	---
F_t	Lateral mooring line peakload or total force	lb lb
$F/\gamma W^2$	Mooring force parameter	---
g	Acceleration of gravity	ft/sec^2
GCLWD	Gulf Coast Low-Water Datum	ft
h	Thickness	ft
H_b	Breaker height	ft
h_o	Height of orbit center above stillwater level	ft

NOTATION

Symbol	Term	Units
hp	Horsepower	ft-lb/sec
H	Design wave height	ft
$H_{1/10}$	Average of highest 10 percent of waves	ft
$H_{1/100}$	Average of highest 1 percent of waves	ft
H_{avg}	Average of all waves	ft
H_b	Breaker wave height	ft
H_i	Incident wave height	ft
H/L	Wave steepness	---
H_{max}	Expected maximum in 500 waves	ft
H_o	Rise of mean level of clapotis formed due to reflecting wave	ft
H_s or $H_{1/3}$	Significant wave height	ft
H_t	Transmitted wave height	ft
i	Maximum trough level	ft
IGLD	International Great Lakes Datum	ft
k_Δ	Layer thickness coefficient	---
K_D	Stability coefficient	---
K_p	Rankine's coefficient of passive earth pressure	---
ℓ	Distance pile penetrates	ft
L	Wavelength	ft
L/W	Ratio of wavelength to breakwater width	---
m	Nearshore slope	---
M	Moment	ft-lb

NOTATION

Symbol	Term	Units
MGL	Mean Gulf level	ft
MLG	Mean low Gulf	ft
MLL	Mean lake level	ft
MLLW	Mean lower low water	ft
mlw	Mean low water	ft
n	Number of layers of armor units	---
N_a	Number of armor units	---
p	Pressure	lb/ft^2
P	Porosity	---
P_i	Power of incident wave train	ft-lb/sec
P_j	Power of hydraulic jets	ft-lb/sec
P_m	Peak impact pressure	lb/ft^2
P_t	Power of transmitted wave train	ft-lb/sec
q	Air discharge	ft^3/sec
r	Thickness of cover or underlayer	ft
R	Resultant force	lb
S_a	Specific gravity of armor unit	lb/ft^3
SWL	Still water level	ft
T	Wave period	sec
U	Velocity of current	ft/sec
W	Width	ft
W_a	Weight of an individual armor unit	lb
W/L	Breakwater width-to-wavelength ratio	---

NOTATION

Symbol	Term	Units
W_t	Design anchor weight	lb
x	Wave reflection coefficient	---
y	Draft	ft
y/d	Relative draft	---
α	Angle of structure slope measured from horizontal	deg
β	Crest width	ft
γ	Unit weight	lb/ft^3
γ_a	Unit weight of armor unit	lb/ft^3
γ_c	Unit weight of concrete in air	lb/ft^3
γ_s	Unit weight of soil	lb/ft^3
γ_w	Unit weight of water	lb/ft^3
μ	Coefficient of soil static friction	---
ρ	Mass density of water	$lb\text{-}sec^2/ft^4$
Φ	Dimensionless horsepower ratio	---
ϕ	Internal friction of sand	---

APPENDIX D

SUMMARIZED INVENTORY OF CORPS OF ENGINEERS

BREAKWATERS AND JETTIES

D-1. The Corps of Engineers (Corps) presently maintains and operates over 600 breakwaters and jetties. The geographical distribution of these structures is summarized in the following tables. Structure types and total lengths by districts within each major coastal area are presented.

Table D-1. Summary of Breakwater and Jetty Types Located on the Pacific Coast

Total Length, Ft, for Indicated Type of Structure

District	Rubble-mound	Timber Pile	Floating	Concrete Panel Wall	Concrete Gravity Structure
Alaska	29,700	1,500	1,100	--	600
Seattle	71,000	7,700	600	--	--
Portland	152,000	--	--	--	--
San Francisco	42,600	--	--	800	--
Los Angeles	108,00	--	--	--	--
Honolulu	12,000	--	--	--	--
Total	415,300	9,200	1,700	800	600

Table D-2. Summary of Breakwater and Jetty Types Located on the Gulf Coast

Total Length, Ft, for Indicated Type of Structure

District	Rubble-mound	Sheet Pile Wall	Concrete Panel Wall	Concrete Gravity
Galveston	169,400	--	1,000	6,600
New Orleans	184,000	--	--	--
Mobile	24,300	--	--	--
Jacksonville	--	1,400	--	--
Total	377,700	1,400	1,000	6,600

Table D-3. Summary of Breakwater and Jetty Types Located on the Atlantic Coast

Total Length, Ft, for Indicated Type of Structure

District	Rubble-mound	Timber Pile	Timber Crib	Steel Sheet Pile wall
New England	39,700	3,900	500	500
New York	18,000	--	--	--
Philadelphia	27,100	600	23,800	--
Baltimore	21,600	--	--	--
Norfolk	4,600	700	--	--
Charleston	66,700	--	--	--
Wilmington	10,400	3,800	--	--
Jacksonville	22,400	--	--	--
Total	210,500	9,000	24,300	500

Table D-4. Summary of Breakwater and Jetty Types Located on the Great Lakes

Total Length, Ft, for Indicated Type of Structure

District	Rubble-mound	Timber Pile	Timber Crib	Steel Wall	Sheet Cell	Pile Bin	Concrete Crib	Concrete Caisson
Buffalo	74,000	--	74,100	--	9,500	1,900	--	--
Detroit	54,000	500	82,600	7,900	5,500	3,200	--	--
Chicago	19,800	--	66,900	--	7,200	--	800	19,800
Total	147,800	500	223,600	7,900	22,200	5,100	800	19,800